LOAD CARRYING CAPACITY OF STONE COLUMNS EMBEDDED IN COMPACTED POND ASH

A Thesis submitted in partial fulfillment of the requirements for the award of the degree

Master of Technology

In

Civil Engineering

(Geotechnical Engineering)

By

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CERTIFICATE

This to certify that the thesis entitled "Load Carrying Capacity of Stone Columns Embedded in Compacted Pond Ash" being submitted by **Jajati Keshari Naik** in the partial fulfillment of the requirements for the award of Master of Technology Degree in **Civil Engineering** with specialization in **GEOTECHNICAL ENGINEERING** at the National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other university/institute for the award of any degree or diploma.

Prof. Suresh Prasad Singh

Place: Rourkela

Date:

Dedicated To

My Father and Mother

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ABSTRACT

Pond ash deposits possess high compressibility, low bearing capacity so acres of land get wasted. Improvement of load carrying capacity of ash ponds will make them suitable for residential or commercial use. Stone or compacted stone columns is a technique of soil reinforcement that is frequently implemented in soft cohesive soils to increase the bearing capacity of the foundation soil, to reduce the settlement, and to accelerate the consolidation of surrounding saturated soft soil. The stress-strain behavior of the granular column is governed mainly by the lateral confining pressure mobilized in the native soft soil to restrain bulging collapse of the granular column.

Several works have been done relating to study the effectiveness of stone column on cohesive material, along with the effect of encasement and without encasement over the stone column. However no studies have been made to explore the effectiveness of stone columns in pond ash deposits. This study relates to the reinforcement of pond ash with stone column and possibility of utilizing abandoned ash pond sites for residential or commercial use.

The purpose of this work is to assess the suitability of reinforcing technique by stone columns to improve the load carrying capacity of pond ash deposits through several laboratory model tests. This objective is achieved in two parts. In the first stage the characterization of pond ash is made along with the evaluation of the mechanical properties like compaction characteristics under different loading conditions, evaluation of shear strength parameters using Direct shear test, Unconfined compression test, Triaxial test at different testing conditions. This is done basically to find out the inherent strength of the pond ash compacted to different densities and at different degree of saturation. In the second series of tests the shear parameters of the compacted pond ash

samples reinforced with stone columns of varying area ratios and length ratios are evaluated from triaxial compression test. In addition to this stone columns having different area ratios and length ratios are introduced in compacted pond ash beds and the bearing capacity of the composite system is evaluated through a series of footing loading tests. For this a circular footing of 75mm in diameter is used.

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NOTATION	DESCRIPTION
E	Compaction Energy, kJ/m3
OMC	Optimum Moisture Content, %
MDD	Maximum Dry Density, gm/cm^3
cu	Unit Cohesion, kg/cm^2
Φ	Angle of Internal Friction, degrees
UCS	Unconfined Compressive Strength, kg/m^2
Cu	Coefficient of uniformity
Cc	coefficient of curvature
G	Specific Gravity
D	Diameter of stone column, cm
q_{ult}	Ultimate bearing capacity
K_p	Coefficient of passive earth pressure

Z	Total depth of the limit of bulge of the pile
γ	Bulk density
V_F	Initial velocity
Nc, N γ, and Nq	Bearing capacity factors
D_f	Depth of foundation
η	wedge angle
ξ	wedge angle
GL	Ground level
Lr	Length ratio

CHAPTER 1

INTRODUCTION

Fly ash is the residue of the coal combustion process in power plants. Nearly 73% of India's total installed power generation capacity is thermal, of which coal based generation is nearly 90 percent (diesel, wind, gas & steam adding to about ten percent). The 85 utility thermal power stations, in addition to several captive power plants, use bituminous or sub-bituminous coal and produce large volumes of flyash. High ash content (30-40%) of Indian coals is contributing to these large volumes of flyash. At present, nearly 170 million tones of flyash is being generated annually in India and nearly 65,000 acres of land is presently occupied by ash ponds. India's dependence on coal as a source of energy shall continue in the next millennium and therefore flyash management would remain an important area of national concern. Its indiscriminate disposal requires large volumes of land, water and energy. Pond ash deposit posses' high compressibility, low bearing capacity so acres of land get wasted. Flyash can be stabilized using compacted stone column to increase the bearing capacity and structures can be built on ash pond in a cost effective manner.

In an era of spiraling land costs and growing population ash pond deposit have been a great headache for the technocrats, administrators, environmentalists and above for the civilization as it results in loss of agriculture production, grazing land and habitat as well as other land use impacts from diversion of Large areas of land to waste disposal. Thousand acres of land occupied by pond ash deposits remains unused as it possess high compressibility and low bearing strength. The use of compacted stone columns as a technique of soil reinforcement is frequently implemented in soft cohesive soils to increase the bearing capacity of the foundation soil, to reduce the settlement, and to accelerate the consolidation of the surrounding saturated soft soil. But very little work has been done on stone column for stabilization of ash ponds. Literature witness that compacted stone column as a stabilizing technique can be applied effectively in silty to fine sand. Flyash also comes in this range. So, in the present study an attempt has been made to study the effectiveness of compacted stone column in improving the bearing capacity of abandoned ash ponds. This objective is achieved in two parts. In the first stage the characterization of pond ash is made along with the evaluation of the mechanical properties like compaction characteristics under different loading conditions, evaluation of shear strength parameters using Direct shear test, Unconfined compression test, Triaxial test at different testing conditions. The effects of saturation on strength parameters also investigated. This is done basically to find out the inherent strength of the pond ash compacted to different densities and at different degree of saturation. In the second series of tests the shear parameters of the compacted pond ash samples reinforced with stone columns of varying area ratios and length ratios are evaluated from triaxial compression test. The area ratios of stone columns are varied from 0 to 40% and the length ratios are varied as 0. 0.25, 0.50, 0.75, and 1.00. In addition to this stone columns having different area ratios and length ratios are introduced in compacted pond ash beds and the bearing capacity of the composite system is evaluated through a series of footing loading tests.

1.1 ORGANIZATION OF THE THESIS

The thesis has been arranged in five chapters as discussed below:

Chapter 1: A brief introduction of the topic is presented

Chapter 2: A detailed literature review is described.

Chapter 3: The experimental work and methodology adopted

Chapter 4: Results and discussion on test results are presented.

Chapter 5: The salient conclusions are reported.

CHAPTER 2

LITERATURE REVIEW

LITERATURE REVIEW

2.1 Introduction

The Use of stone column as a ground improvement technique is of recent origin. Stone columns are extensively used to improve the bearing capacity of poor ground, time rate of settlements, stiffness, shear strength of soil and can also be used to reduce the settlement of structure, liquefaction potential of soft ground. The stone column technique is widely used to strengthen the ground so as to support various geotechnical facilities like embankments, oil tanks on poor ground, low-rise buildings, highway facilities, bridge abutments. The method is generally adopted in clayey soils. Various researchers have worked on stone columns. Many numerical analyses, model tests, field tests, mathematical simulations are carried out to study the effects of stone columns on poor ground. However the design of stone columns till date is based on the empirical approach as the load settlement behavior of stone columns are influenced by a number of factors. The available literature on stone column is discussed in this chapter.

2.2 Methods of Installation of stone columns

The Use of stone column as a ground improvement technique is of recent origin. The method is generally adopted in clayey soils. This can be treated as the extension of technique of densification of cohesion less soil by vibrofloat. Earlier stone columns were formed by vibrofloat but now they are also formed by forming a bore as in bored cast in situ concrete piles. The primary purpose of soil improvement by stone column technique is mainly to increase the bearing capacity of foundation soil and also to reduce post construction settlement. The method has been mainly used to improve subsoil below buildings, embankments. Stone columns are constructed using either vibro-replacement or vibro-displacement methods.

2.2.1 Vibro-replacement method

Vibro-replacement is a ground improvement technique that constructs stone columns by means of a crane-suspended down hole vibrator, to reinforce all soils and densify granular soils. Vibro replacement stone columns are constructed with either the wet top feed process, or the dry bottom feed process.

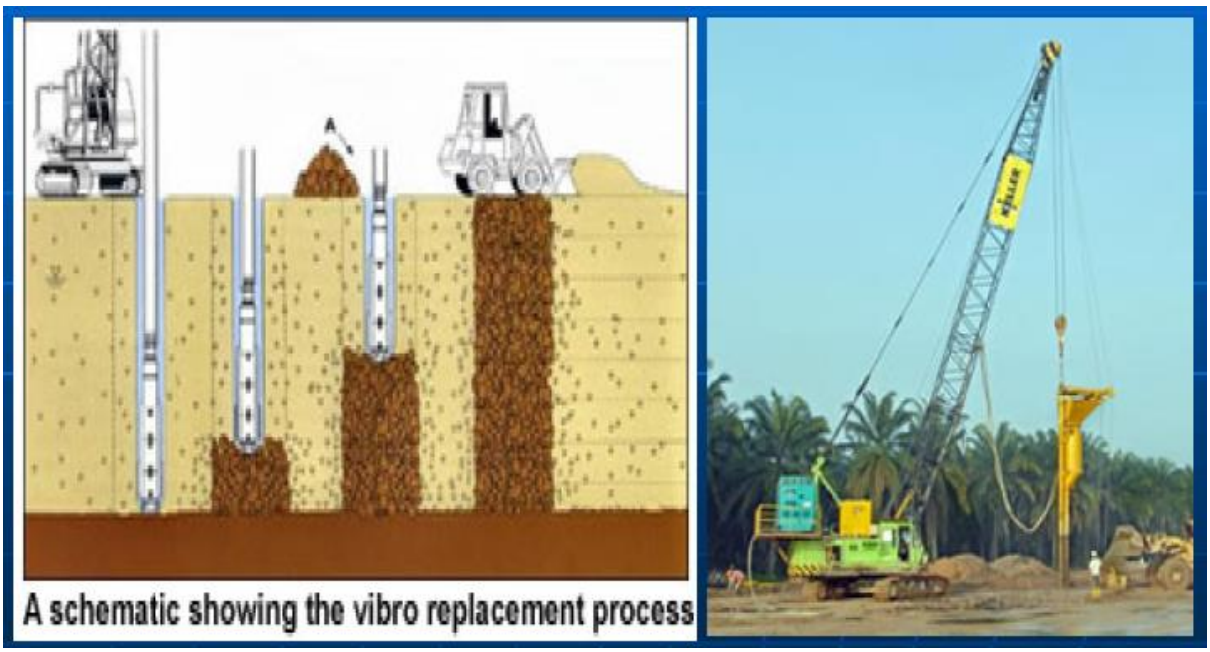


Fig-2.1: Vibro-replacement process

2.2.1.1 Wet top feed process

In the wet top feed process, the vibrator penetrates to the design depth by means of the vibrator's weight and vibrations, as well as water jets located in the vibrator's tip. The stone (crushed stone or recycled concrete) is then introduced at the ground surface to the annular space around the vibrator created by the jetting water. The stone falls through the annular space to the vibrator tip, and fills the void created as the vibrator is lifted several feet. The vibrator is lowered, densifying and displacing the underlying stone. The vibro replacement process is repeated until a dense stone column is constructed to the ground surface.

Fig-2.2: Wet top feed process

2.2.1.2 Dry bottom feed process

The dry bottom feed process is similar except that no water jets are used and the stone is fed to the vibrator tip through a feed pipe attached to the vibrator. Pre drilling of dense strata at the column location may be required for the vibrator to penetrate to the design depth. Both methods of construction create a high modulus stone column that reinforces the treatment zone and densifies surrounding granular soils.

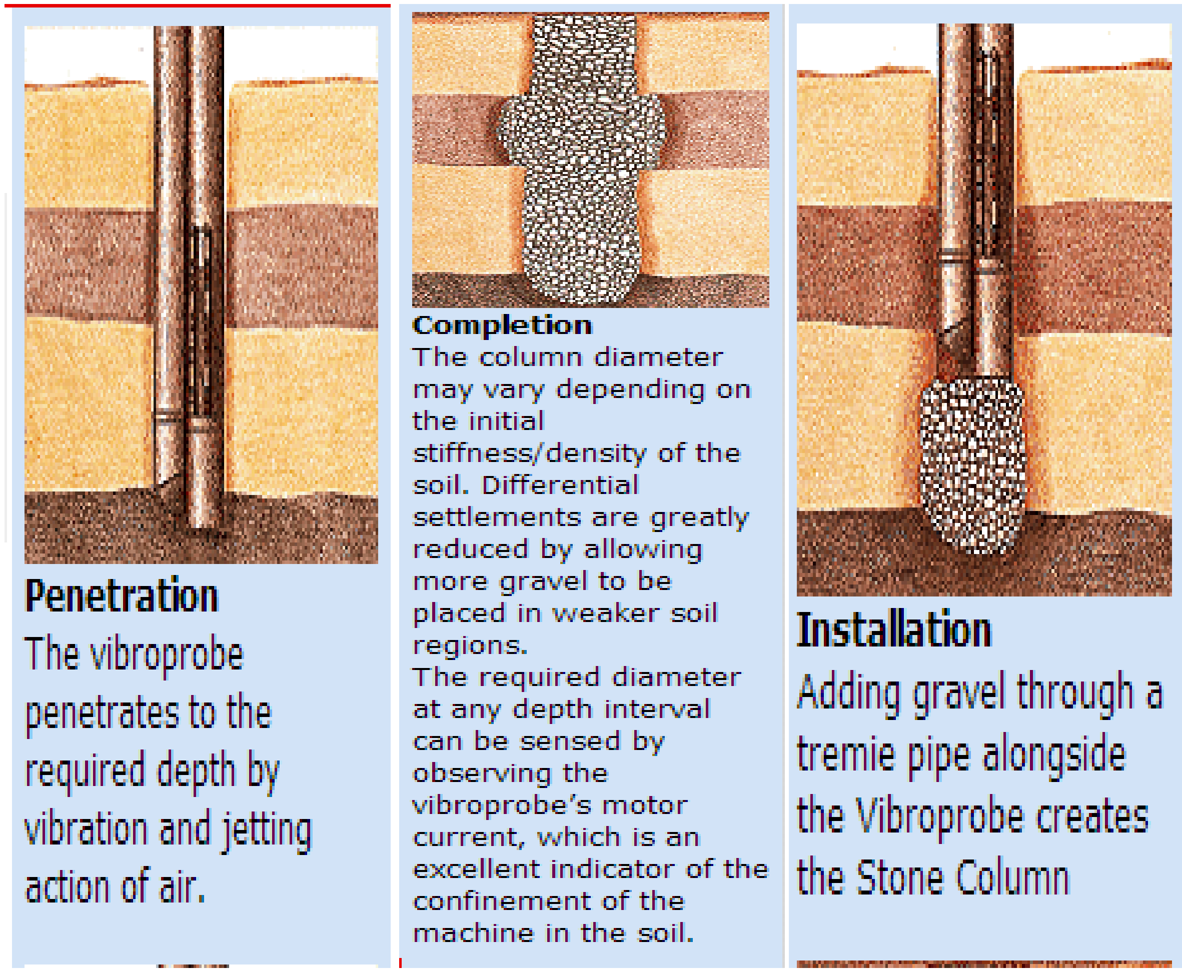


Fig-2.3: Dry bottom feed process

2.2.2 Bored piling technique

This method has been developed in India has been gaining importance. A cased hole of required size is bored using conventional tools such as flap valve bailer and casing tube of required size. After the casing tube is driven to required depth, granular fill material is filled. Tube is withdrawn by short pass as required and granular fill compacted by rammer. The filling of the granular material, withdrawal of the casing tube and ramming of fill is so controlled as to have continuous column of stone column. Compaction is achieved by a rammer generally of 1.5 to 2tonnes and falling through a height of 1 to 1.5 m.

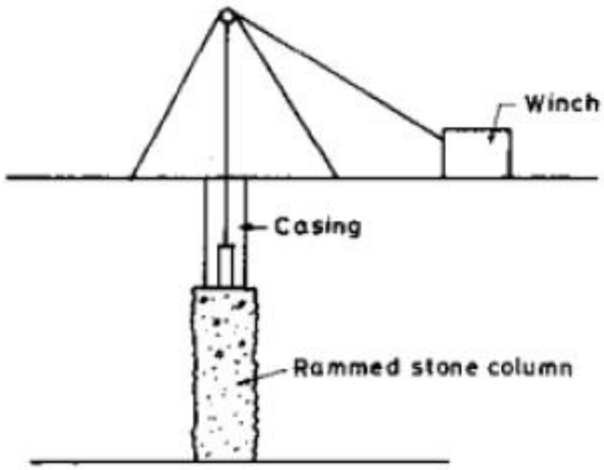


Fig-2.4 Cased rammed stone column

2.3 Design Concept

It is true that design of stone column is less understood but it is as empirical as the design of pile foundation. A stone column derives its support basically from lateral resistance provided by the surrounding soil to the expansion caused by bulging of the un cemented stone column under the load.

The important parameters in estimating the capacity of stone column are

a. Angle of internal friction of the column material
b. Diameter of the stone column formed and
c. Un drained shear strength of surrounding soil
d. In-situ lateral stress in the soil
e. Radial pressure /deformation characteristics of the soil

The angle of internal friction depends on the material type, its gradation and shape and effectiveness of compaction. Generally angle of friction obtained is between38^0 to 55^0. Higher angle can be adopted for the rammed stone columns that for the vibrated ones.

2.4 Suitable soils

The soil which does not respond to vibration alone is good for stone column. They are silty and clayey sands, silts, very fine sands, clays and some layered soils. The effectiveness of stone columns in different types of soil is given in Table 2.1.

Table 2.1 Expected vibro-replacement stone column results

Ground type	Relative effectiveness	
	Densification	Reinforcement
sands	excellent	very good
silty sands	very good	very good
non plastic silts	good	excellent
clays	marginal	excellent
mine spoils	excellent depending on gradation	good
dumped fill	good	good
garbage	not applicable	not applicable

2.5 Failure mechanism of stone column:

The possible modes of failure of stone columns are:

- Bulging Failure
- Pile Failure
- General Shear Failure

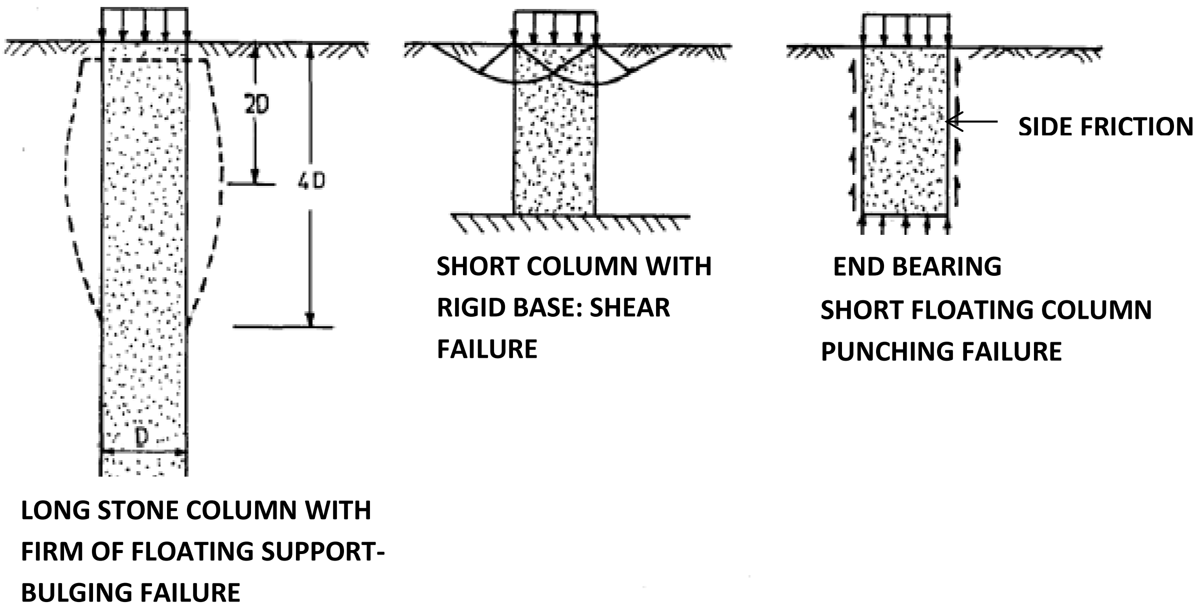


Fig-2.5 Failure mechanism of single stone column in a homogenous soft layer

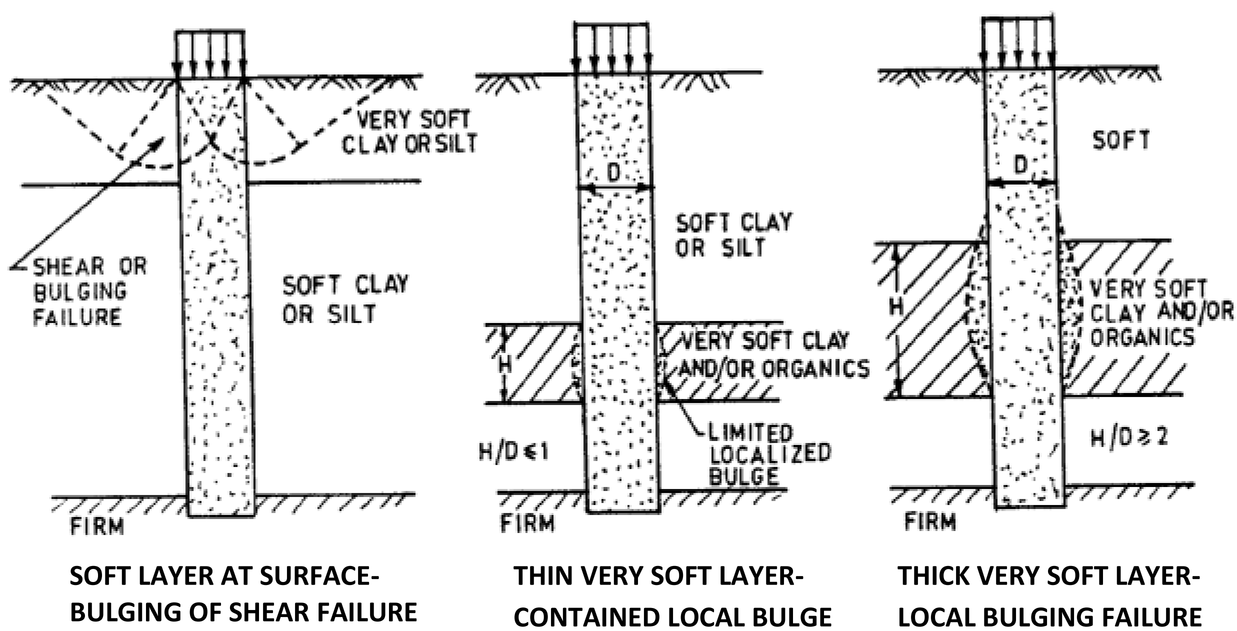


Fig-2.6 Failure mechanism of single stone column in a non-homogenous soft layer

2.6 Ultimate Bearing Capacity of Single Granular Pile

A realistic assessment of the ultimate bearing capacity of the supporting soil is of paramount importance for safe and economic design of the foundation. During the last three decades or more, efforts have been made by investigators all over the world to provide a solution to the problem of ultimate bearing capacity through experimental and analytical techniques. The various approaches are:

a. Passive pressure or plastic failure approach
b. General shear failure approach

2.6.1 Based on Passive Pressure Approach

In the passive pressure approach, the load applied through a strip footing on a granular pile top tends to concentrate on the granular pile which is the stronger material of the composite foundation soil. The pile material dilates and exerts lateral stresses on the surrounding clay which are resisted by the passive earth pressure. Conventional theory of passive pressure implies an increase of pressure with depth. There will be a zone of no significant deformation within the pile under the rigid concrete footing. It was the belief that the ultimate lateral strength of the single granular pile is equal to the ultimate lateral strength of the soil surrounding the pile. Thus ultimate bearing capacity of a granular pile is given by the following equation as a two dimensional plastic failure case

$$\mathbf{q_{ult} = Pp = \gamma Z K_p + 2c_u \sqrt{K_p}} \quad \ldots\ldots(2.1)$$

Where q_{ult} is the ultimate load bearing capacity of the granular pile, γ the bulk density of clay, Z the total depth of the limit of bulge of the pile and K_p is the coefficient of passive earth pressure.

The total depth of bulge Z is equal to the depth of the footing from the ground level plus the depth of the bulge of the pile which is critical pile length. In case of a clay of essentially uniform strength, the passive restraint just below the dotted line, the granular pile will be the weakest where the lateral support is the least which is about 1.75m to 2m below ground level. This critical length is found to be equal to 2 times the pile diameter. However , in case of bulging failure mode in clay, the critical length is found to be 4 times the pile diameter .The ultimate bearing capacity determined from equation (1) is conservative estimate of granular pile capacity. The lateral passive restraint on the pile away from the edge of loaded area under the wide spread footing is much larger due to equal all round pressure influence due to surcharge load. Thus the total carrying capacity of the granular pile increase until the local shear failure in clay (due to contact stresses with the individual pile material back fill particles) or the end bearing failure of the pile whichever occurs earlier. The ultimate bearing capacity of the pile, q_{ult} depends on its diameter and is given by following equation.

2.6.2 General Shear Failure Approach

Madhav and Vitkar (1979) stipulated the plain strain version of a granular pile as a granular trench and postulated the failure mechanism. Utilizing limit analysis approach, an analytical solution has been developed.

Using the upper bound theorem, the work equation is formed by equating the external rate of work done due to (a) external applied load (b) soil weight and (c) soil surcharge, to the internal energy dissipated in the plastically determined region, for which Coulomb's yield criterion is valid.

The general shear failure mechanism is postulated for two cases A/B≤1 and A/B≥1 (Fig. x), where A is trench width and B is width of strip footing resting on soil trench system with the foundation at a depth D_f.

The different zones are

- an active Rankine zone AGC with wedge angle ξ and
- a mixed transition zone GCD with central angle θ_1 bounded by long spiral based on frictional angle, Φ_1 of trench material.
- a transition zone GDE with a central angle θ_2 bounded by log spiral based on frictional angle, Φ_2 of the weak clay.
- a passive Rankine zone GEF with wedge angle η

The wedge AGC of active rank moves vertically down as a rigid body with the same initial velocity V_F of the footing. The downward movement of the footing and wedge AGC is accommodated by the lateral movement of the adjacent soil. The central angle θ_1 and θ_2 depend upon wedge angle ξ and η, the ratio A/B and the angle of internal friction Φ_1 of the trench material. The properties of the granular trench material considered are cohesions, c_1, angle of internal friction of trench material, Φ_1 and density of trenching material, γ_1. Cohesion c_1 of trench material could be zero. However the theory is developed for the most general case of c-Φ- γ soil. The properties of natural soil are cohesion c_2, angle of internal friction Φ_2 and density γ_2.

From the geometry of the failure surfaces, the lengths and velocities at various discontinuities are found. The rate at which the work is done by soil weight is found by multiplying the area of each rigid body by γ times the vertical component of the velocity of the rigid body. The velocity component of the zone AGC, GCD, GDE and GEF are considered to act in the same direction at that of the force V_F, while that of surcharge in opposite direction. This convention is based on whether the work is done against V_F or in the same direction as that of $V_{F.}$

The work equation is formulated by equation total rate which the work is done by (a) external load on the foundation (b) soil weight in motion and (c) the surcharge to total rate of energy dissipation. Equating work done by external load, qult, to the energies dissipated by cohesion and work done on account of soil weight and surcharge , equation x is obtained.

$q_{ult} = c_2 N_c + (\gamma_2 B/2) N_\gamma + \gamma_2 D_f N_q$

Where $N_c = [c_1/c_2] N_{c1} + N_{c2}$

And $N\gamma = [\gamma_1/\gamma_2] N_{y1} + N_{\gamma 2}$

Nc1, Nc2, N $\gamma_{1,}$ N γ_2 and Nq are dimensionless factors, depending upon the properties of trench, soil material and ratio of A/B.

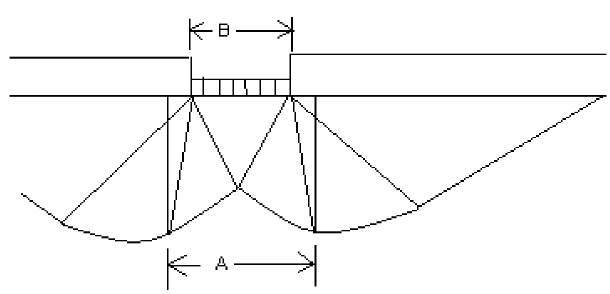


Fig. 2.7 Mechanism of general shear failure (A/B≤1)

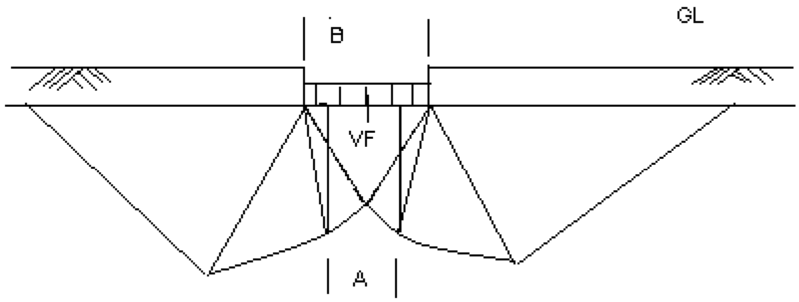


Fig. 2.8 Mechanism of general shear failures (A/B≤1)

Advantages

• Stone Columns are designed to reduce settlements of compressible soil layers in orde rto be able to build most structures with shallow footings and slab-on-grades on very soft soil;

• When applicable, their draining characteristics result in an increase in the time rate of consolidation settlement in soft cohesive soil;

• Because they are made of compacted granular material, no curing period is necessary and no cut-off to the shallow footing grades are required as the excavation of the footing can immediately follow the installation of the stone columns down to the required elevation;

• High production rates;

• Stone Columns are also well-adapted to the mitigation of liquefaction potential thanks to the combined effect/advantage of their draining potential and the increase of shear strength and stiffness of the improved soils.

Application

- Industrial warehouses and commercial buildings ;
- Condominium, apartment buildings, townhouses and single-family residential developments;
- Reclaimed platforms (harbours, container terminals);
- Sewage treatment plants;
- Railway and roadway embankments;
- Retaining walls;
- Liquefaction mitigation and building support in seismic areas.

2.7 Load settlement behavior of stone column

Various researchers have worked on stone columns. These works mainly focus on evaluation of load carrying capacity and settlement analysis of soft ground reinforced with stone columns. All these works can be grouped under the following sub headings:

- Numerical and Analytical Studies
- Theoretical Analysis
- Model tests
- Prototype/ Field tests

2.7.1 Numerical and Analytical Studies

Guetif et al. (2007) proposed a method for evaluating the improvement of the Young modulus of soft clay in which a vibro-compacted stone column is installed. A composite cell model is considered and numerical analysis is carried out using PLAXIS software to simulate the vibro-compaction technique that leads to a form of primary consolidation of the soft clay. Mohr

Coulomb perfect plastic behaviour is considered for the numerical simulation to the improved soil constituents. The degree of improvement of the Young modulus of soft clay has been estimated from numerical results and the zone of influence of the improved soft clay has been predicted.

Deb and Dhar (2011) Proposed a combined simulation-optimization-based methodology to identify the optimal design parameters for granular based stone column improved soft soil. The methodology combines a finite difference based simulation model and an evolutionary multi objectives optimization model. For minimization of maximum settlement and minimization of differential settlement subjected to stress constraints and maximization of degree of consolidation subjected to stress constraints a combined optimization simulation technique is used. It shows that modular ratio and ultimate stress carrying capacity of stone column are the two important parameters for optimal design.

Castro and Sagaseta (2011) Performed a coupled of finite element analysis of the consolidation and deformation around stone column to assess the accuracy of different analytical solution. A simple elastic or elasto-plastic soil models are used and surface settlement, dissipation of pore pressure, vertical stress concentration are studied. Soil responses are estimated including the radial and plastic strain in the column.

Elshazly et al. (2008) Studied the relation between the inter column spacing and corresponding alteration of soil state of stress is found out. A case history, involving three columns patterns along with the irrelevant field and laboratory test results, is utilized and a well-tested finite element model is employed in the analysis.

Zahmatkesh and Choobbasti (2012) Evaluated the settlement of soft clay reinforced with stone column and finite element analyses are carried out using 15 noded triangular elements with

PLAXIS. A drained analysis is carried out using Mohr–Coulomb's criterion for soft clay, stones, and sand. The settlement ratio (SR) is evaluated using secant modulus and it is found that SR decreases with compaction of surrounding soft soil. It is mainly due to a stiffer column material.

Deb. (2008) used a mechanical model for predicting the behaviour of granular bed-stone column-reinforced soft ground. The granular layer placed over the stone column is reinforced soft soil is characterized by Pasternak shear layer. The saturated soft soil is idealized by the Kelvin–Voigt model to represent its time-dependent behaviour and the stone columns are idealized by stiffer Winkler springs. It is observed that presence of granular bed on the top of the stone columns helps to transfer stress from soil to stone columns and reduces maximum as well as differential settlement.

Lee and Pandey (1998) Proposed a numerical model to analyse elastic as well as elasto plastic behaviour of stone-column reinforced foundations. The model is implemented in an axi-symmetric finite element code and numerical prediction is made for the behaviour of model circular footing resting on stone column reinforced foundation.

2.7.2 Theoretical Analysis

Maheshwari and Khatri (2011) represent a constitutive relation in which granular fill layer, soft soil and stone columns are represented by Pasternak shear layer, Kelvin-Voigt body and Winkler springs respectively. Non linear behaviour of these is considered by means of constitutive relationships.

Adalier and Elgamal (2004) studied the reduction in liquefaction and associated ground deformation using stone column.

Christoulas et al. (1997) studied the reinforcing effect of stone column on the stability of road embankment. Stability analysis of stone column and discrete soil were carried out and the results are compared with the results of the analyses based on DiMaggio's approach.

Babu et al. (2013) discussed the techniques, methods of construction of stone column, mechanisms of stone column behaviour under load and associated design philosophies along with some practical problems.

Najjar (2013) assembled the published results from field, laboratory, and numerical investigations of sand/stone columns in clay in which focus is on the modelling, testing, and analysis of soft clays that are reinforced with sand/stone columns in relation to bearing capacity and settlement considerations.

2.7.3 Model studies

Castro et al. (2012) studied the consolidation and deformation around end bearing columns under distributed loads and compared the laboratory results with analytical solution and numerical simulation. Equivalent coefficient of consolidation, stress concentration factors and settlement reduction are analysed. Soil improvement is directly dependent on the stress distribution between the soil and column. Column yielding, friction and dilatancy angle of gravel influence the final improvement.

Deb et al. (2011) presented a series of model tests on unreinforced and geogrid reinforced sand bed resting on stone column. The load carrying capacity of soft soil, depth of bulge of stone column increases and bulge diameter decreases due to the placement of sand bed and it is more beneficial in sand bed reinforced with geogrid.

Shivashankar et al. (2010) studied the improvement in load carrying capacity, stiffness, resistance to bulging of stone column installed in soft soil due to a series of laboratory plate load test. Vertical nails are inserted along the circumference of stone column and it is found that stone column reinforced with nails has higher load carrying capacity, lesser compression and lesser lateral bulging. It is also observed that the benefit of nails increases with increase in diameter, number and depth of embedment of the nails.

Shivashankar et al. **(2011)** studied the behaviour of stone column in layered soil consisting of weak soil in the top layer under a series of plate load tests. The entire area in the unit cell tank is loaded and stiffness of improved ground is estimated. Secondly the stone column is loaded and axial capacity is determined. It is found that the depth of top weak soil layer has a great influence on stiffness, load bearing capacity and bulging of stone column.

Frikha et al. (2013) presented the behaviour of remoulded kaolin clay reinforced by stone column. It is found that Young's modulus of kaolin clay increases as the cavity expansion ratio and consolidation stress increases and the undrained shear strength is more at lower at consolidation stress. It is also noted that the ratio of undrained Young's modulus to undrained shear stress increases when the consolidation stress decreases.

Vekli et al. (2012) studied the effect of stone columns (SCs) and s/D ratio (distance between the vertical axes of SCs/diameter of SCs) on slope stability, bearing capacity and settlement using small scale laboratory model and its numerical model. For various slope PLAXIS is used to analyse the investigation. It is observed that the bearing capacity increases and settlement decreases due to the insertion of stone columns. Comparison is done on experimental tests and finite element analysis.

2.7.4 Prototype/ Field tests

Poorooshasb and Meyerhof. (1997) studied the efficiency of end bearing stone column and lime column in reducing the settlement of foundation system and showed the various factors like stone column spacing, weak soil properties, properties of granular medium, in situ stress caused by the installation technique, magnitude of the load carried by the supported raft foundation that influence the stone column behaviour.

Kumar (2001) Evaluated the reduction in liquefaction potential due to dynamic compaction and construction of stone columns. Construction of stone columns densified the soil to required depth and helped to support a five storey building constructed on strip and spread shallow foundation.

SCOPE OF THE PRESENT STUDY

Due to non-availability of land near to the thermal power plants or land cost it became imperative to plan optimum utilization of the available land. In this context utilization of abandoned ash ponds has gained importance. However, due to low bearing capacity and high compressibility of the ash ponds construction activities on pond ash is not possible. Hence, in this work an attempt has been made to investigate the effectiveness of stone columns in modifying the stress-strain response of the pond ash deposits.

SCOPE:

- To characterize the pond ash and find out the effects of compaction energy on strength and compatibility pond ash
- To study the stress-strain response of compacted pond ash reinforced with stone columns of different area ratios and length ratios
- To find out the bearing capacity and settlement response of pond ash beds reinforced with stone columns.

CHAPTER 3

EXPERIMANTAL WORK

AND

METHODOLOGY

EXPERIMANTAL WORK AND METHODOLOGY

3.1 INTRODUCTION

Nearly 65,000 acres of valuable land is occupied by ash ponds. The pond ash deposits are characterized by its very low bearing capacity and high compressibility, rendering them unsuitable for any civil engineering structures constructed over it. Any construction activity over abandoned ash ponds needs a proper understanding of the physical and mechanical properties of these deposits and also the suitability of any ground improvement techniques that can be adopted. Even through adequate substitute for full scale field tests are not available; tests at laboratory scale provide a means to closely control many of the variable encountered in practice. The trends and behavior pattern observed in the laboratory tests can be used in understanding the performance of the structures in the field and may be used in formulating mathematical relationship to predict the behavior of field structures. Keeping this in mind laboratory investigations were carried out to determine the physical and mechanical properties of pond ash. In addition to this the suitability of stone columns in improving the load carrying capacity pond ash deposits were examined through a series of model tests. This chapter outlines experimental work undertaken, the methodology adopted and the salient test results.

3.2 MATERIAL USED

3.2.1 Pond ash

Pond ash was collected from ash ponds of Rourkela Steel Plant (RSP) Rourkela. The sample was sieved through 2mm sieve to separate out the foreign and vegetative matters. The collected samples were mixed thoroughly to get the homogeneity and oven dried at the temperature of 105-110^{0}C. The pond ash samples were stored in airtight container for subsequent use.

3.2.2 Stone Aggregates

Screened stone aggregates were obtained from local crusher. All these aggregates were washed and oven dried at the temperature of 110^{0}C degree. The stone aggregates were stored in airtight container for subsequent use and protected from water moisture. The dried aggregates of two size having particle size between 2mm to 4mm and 1mm to 2mm were used for preparation of the stone column.

3.3 TESTING PROGRAM

Two series of tests were carried out in this work. The first series of tests aimed at evaluating the physical and mechanical properties of pond ash which includes the index properties of pond ash such as the specific gravity, grain size distribution and the consistency indices. Further the compatibility of pond ash under different compactive energy levels was determined with the help of compaction tests. The shear strength parameters of compacted pond ash specimens at OMC and saturation conditions were also determined from direct shear test and triaxial shear tests.

The second series of tests were carried out to evaluate the reinforcing effects of stone columns in improving the load carrying capacity of compacted pond ash samples. The stress strain response of pond ash reinforced with stone was determined by triaxial test. Under the triaxial test the compacted pond ash was reinforced with stone column by varying diameter of stone columns as 2.2cm, 2.6cm, 3.5cm, and 4cm to maintain the area ratio of 10%, 20%, 30%, and 40%. The effectiveness of length of stone columns on triaxial behaviour of samples was studied by varying the length ratio as 1.00, 0.75, 0.5, and 0.25. All these specimens were of 75mm diameter and 150mm in length. These samples were tested in a triaxial testing machine with cell pressure varying as 0, 1, 2, and 3 kg/cm^2 with axial strain rate of 1.25%. Further work has done to evaluate the bearing capacity of compacted pond ash beds reinforced with stone column. The stone columns with diameters of 2.6cm, 3.3cm, 4.8cm and 5.7cm were installed in the pond ash bed which corresponds to area ratio of 10%, 20%, 40% and 60%. The effectiveness of length of stone columns on bearing capacity of pond ash beds was studied by varying the length ratio as 1.00, 0.75, 0.5, and 0.25. The details of tests conducted and the experimental procedure are outlined below.

3.4 DETERMINATION OF INDEX PROPERTIES

3.4.1 Determination of specific gravity

The specific gravity of pond ash was determined according to IS: 2720 (Part-III, Section-1, 1980). The specific gravity of pond ash was found to be 2.30.

3.4.2 Determination of grain size

For determination of grain size distribution, the pond ash was passed through an IS test sieve having an opening size 75µ. Sieve analysis was conducted for coarser particles as per IS: 2720 part (IV), 1975 and hydrometer analysis was conducted for finer particles as per IS: 2720 part (IV) The percentage of pond ash passing through 75 µ sieve was found to be 82.4%. Hence almost all the pond ash particles are silt size. Coefficient of uniformity (C_u) and coefficient of curvature (C_c) for pond ash are 6.13 and 2.61 respectively

3.5 DETERMINATION OF ENGINEERING PROPERTIES

3.5.1 Compaction characteristics of pond ash

The compaction characteristics of pond ash was found by using compaction tests as per IS: 2720 (Part VII) -1980 and IS: 2720 (Part VIII)-1980. For this test, samples were mixed with required amount of water and the wet sample was compacted in Proctor mould of 1000c.c volume, either in three or five equal layers using standard Proctor rammer of 2.6 kg or modified Proctor rammer of 4.5 kg. The number of blows in each layer is adjusted so as to impart energy of 119, 357, 595, 593, 1604 and 2674 kJ/m3 of compacted volume. The moisture content of the compacted mixture was determined as per IS: 2720 (Part II) 1973. From the dry density and moisture content relationship, optimum moisture content (OMC) and maximum dry density (MDD) were determined. The test results are given in Table 3.1.

3.5.2 Determination of Shear Parameters

The Direct shear test is one of the common tests used to study the strength parameter of soil. To get the strength parameter, Direct shear tests on pond ash specimens compacted to their corresponding MDD at OMC with compactive effort varying as 119, 357, 595, 1604, 2674kJ/m^3 were performed according to IS: 2720 (Part X)-1991. For this test specimens were prepared corresponding to their MDD at OMC in the metallic split mould with dimension 60mm (breadth) × 60mm (width)× 26mm(height). These specimens were tested in a direct shear testing machine with strain rate of 1.25 mm/minute till failure of the sample. The test results are given in Table 3.1. To study the effectiveness of shear parameter under saturation condition the same making and testing procedure of sample specimen was followed as above only the water has poured over the sample specimen for 30 minute to make the sample saturate. The test results are given in Table 3.1

Table 3.1 Variation of OMC, MDD and shear parameters at different compaction level

Compaction energy(kJ/m³)	OMC (%)	Dry density(gm/cm^3)	C at OMC (kg/cm^2)	C at saturation (kg/cm^2)	ϕ at OMC	ϕ at saturation
119	43.23	0.984	0.13	0.018	22	16
357	41	1.031	0.14	0.11	27	25
595	35.5	1.134	0.17	0.13	29.46	27
1604	32.22	1.15	0.21	0.16	36.86	33.45
2674	31.7	1.23	0.25	0.2	38.6	36.76

$$\begin{bmatrix} \phi = \text{Angle of internal friction in degree} \\ C = \text{unit cohesion in kg/cm}^2 \end{bmatrix}$$

3.5.3 Determination of Unconfined Compressive Strength

The Unconfined compressive strength test is one of the common tests used to study the strength characteristics of soil and stabilized soil. To get immediate UCS strength, UCS tests on pond ash specimens compacted to their corresponding MDD at OMC with compactive effort varying as 119, 357, 595, 1604, 2674 kJ/m^3 were performed according to IS: 2720 (Part X)-1991. For this test cylindrical specimens were prepared corresponding to their MDD at OMC for particular compaction energy. The specimen was prepared in metallic cylindrical mould with dimension 50mm (dia.) × 100mm (high) as shown in Fig3.1. These specimens were tested in a compression testing machine with strain rate of 1.25 mm/minute till failure of the sample. The test results are given in Table 3.3. To find the effect of saturation on strength of pond ash specimen were wax coated and water is allowed to percolate from the top surface till the specimen gets saturated and tested (Fig3.2). The test result are presented in Table 3.4


Fig no- 3.1 Compacted pond ash specimen for UCS tests


Fig no-3.2 Compacted pond ash covered with wax

Table 3.2 UCS values and failure strains of pond ash specimens compacted at OMC

Compaction energy(kJ/m³)	119	357	595	1604	2674
Stress in kPa	19.587	30.365	48.446	58.413	66.758
Strain in %	2.75	2.5	2.5	2.25	2.25

Table 3.3 UCS values and failure strains of pond ash specimens at saturation condition

Comapaction energy(kJ/m³)	119	357	595	1604	2674
Stress in kPa	8.142	18.67	26.98	32.78	38.45
Strain in %	2.5	3	3	3	3

3.5.4 Triaxial tests on compacted pond ash

The traiaxial test was conducted to study the stress-strain response of pond ash under different confining pressure. The tests were conducted at densities of 0.984, 1.031, 1.134, 1.15, and 1.23gm/cm³ which were obtained from compaction tests corresponding to compaction energies of 119, 357, 595, 1604, and 2674kJ/m³. The test specimens were of 50mm (dia.) × 100mm (high) in size. The traiaxial test was conducted very carefully at the confining pressure of 1 kg/cm^2, 2 kg/cm^{2,} and 3 kg/cm^2. The test result are presented in Table 3.5

Table 3.4 Triaxial shear test results of unreinforced compacted pond ash samples

Energy in kJ/m³	Confinement Pressure (kg/cm^2)						Unit cohesion (kg/cm^2)	Angle of internal friction (degrees)
	3		2		1			
	Stress (Kg/cm²)	Strain (mm)	Stress (Kg/cm²)	Strain (mm)	Stress (Kg/cm²)	Strain (mm)		
2674	8.15	0.45	5.55	0.57	3.05	0.47	0.239	37.4
1604	7.17	0.7	4.98	0.62	2.63	0.325	0.18	32.376
595	5.94	0.6	3.97	0.55	2.02	0.3	0.147	28.32
357	5.87	0.85	4.09	0.87	2.02	0.7	0.114	25.63
119	4.7	0.6	4.7	0.6	1.67	0.625	0.106	19.87

3.5.5 Determination of maximum density of stone aggregate

The vibration test was conducted to get the maximum density of stone aggregate having the size ranges of 1mm to 2mm and 2mm to 4.75mm size aggregate with varying the mixing proportion of aggregate. The test results are presented in Table 3.6. The shear strength parameters of the stone aggregate at mass density of 1.824 gm/cm^3 was found to be C=0 and Φ=45^0 and this proportion is used to prepare the stone columns.

Table 3.5: Different Density of Stone Aggregate in Different Mixing Proportion

Grade Size	Mixing Proportion (%)	Dry Density (gm/cm^3)
4.75-2mm & 2mm-1mm	50+50	1.824
4.75-2mm & 2mm-1mm	60+40	1.756
4.75-2mm & 2mm-1mm	40+60	1.766
4.75-2mm	100	1.716
2mm-1mm	100	1.609

3.6 TEST ON STONE COLUMNS REINFORCED SAMPLES

3.6.1 Trixial shear test on compacted pond ash reinforced with stone columns

The traiaxial test was conducted to study the response of reinforced pond ash. The compacted pond ash samples were prepared at dry densities of 0.90 g/cm³ or 0.984 g/cm³. These samples were of 75mm (dia.) ×150mm (height). Four different thin tubes of external diameters of 2.2cm, 2.6cm, 3.5cm, and 4.0cm were used to make cavity at the center of cylindrical sample to give the area ratio of 10%, 20%, 30% and 40%. At the center of pond ash sample stone aggregate are inserted and compacted with compaction instrument to maintain the density of stone aggregate. To study the effect of length ratio the lengths of the stone columns were adjusted to give length ratio of 0.25, 0.50, 0.75, and 1.0. Traiaxial test was conducted by give the confining radial cell pressures of 3 kg/cm^2, 2 kg/cm^2, 1 kg/cm^2 and 0 kg/cm^2 and the test result are presented in Tables 3.7 to 3.14. The apparatus and tools used to make the sample and stone column are were shown in Fig 3.3 and Fig 3.4

Table 3.6 Triaxial shear test results for reinforced (compacted density of 0.90 g/cm^3) pond ash samples at confining pressure of 3 kg/cm^2

Stone column dia	CONFINEMENT PRESSURE At 3 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0 cm	8.784							0.17
2.2 Cm	10.89	0.131	10.23	0.137	9.86	0.14	9.41	0.123
2.6 Cm	12.87	0.123	12.39	0.129	11.12	0.136	10.25	0.145
3.5 Cm	14.21	0.121	13.54	0.126	12.56	0.131	11.85	0.125
4.0 Cm	15.87	0.185	14.56	0.174	13.52	0.163	12.89	0.188

Table 3.7 Triaxial shear test results for reinforced (compacted density of 0.90 g/cm^3) pond ash samples at confining pressure of 2 kg/cm^2

Stone column dia	CONFINEMENT PRESSURE At 2 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0 cm	5.765							0.178
2.2 Cm	7.86	0.153	7.26	0.157	6.78	0.151	6.12	0.105
2.6 Cm	9.149	0.128	9.038	0.137	7.84	0.148	6.94	0.155
3.5 Cm	10.24	0.126	9.187	0.132	8.76	0.143	7.89	0.145
4.0 Cm	11.25	0.171	10.46	0.184	9.312	0.191	8.985	0.196

Table 3.8 Triaxial shear test results for reinforced (compacted density of 0.90 g/cm³) pond ash samples at confining pressure of 1 kg/cm^2

Stone column dia	CONFINEMENT PRESSURE At 1Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0 cm	2.89							0.188
2.2 Cm	4.08	0.163	4.25	0.159	3.96	0.155	3.48	0.116
2.6 Cm	4.85	0.143	5.21	0.147	4.56	0.156	3.89	0.161
3.5 Cm	5.68	0.145	5.86	0.146	4.89	0.151	4.24	0.15
4.0 Cm	6.08	0.195	6.21	0.181	5.21	0.175	4.94	0.203

Table 3.9 Triaxial shear test results for reinforced (compacted density of 0.90 g/cm³) pond ash samples at confining pressure of 0 kg/cm^2

Stone column dia	CONFINEMENT PRESSURE At 0 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0 cm	0.156							0.057
2.2 Cm	0.192	0.138	0.179	0.132	0.165	0.14	0.159	0.123
2.6 Cm	0.125	0.123	0.169	0.127	0.161	0.136	0.149	0.145
3.5 Cm	0.086	0.121	0.097	0.119	0.129	0.097	0.147	0.125
4.0 Cm	0.071	0.185	0.081	0.174	0.114	0.081	0.126	0.188

Table 3.10 Triaxial shear test results for reinforced (compacted density of 0.984 g/cm³) pond ash samples at confining pressure of 3 kg/cm^2

Area ratio (%)	CONFINEMENT PRESSURE At 3 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0	9.894							0.17
10	11.224	0.124	11.134	0.128	11.084	0.135	10.281	0.119
20	12.388	0.117	12.246	0.119	12.162	0.126	11.073	0.098
30	14.652	0.097	13.517	0.112	12.337	0.121	11.97	0.084
40	16.815	0.081	14.652	0.073	13.426	0.061	13.045	0.054

Table 3.11 Triaxial shear test results for reinforced (compacted density of 0.984 g/cm³) pond ash samples at confining pressure of 2 kg/cm^2

Area ratio (%)	CONFINEMENT PRESSURE At 2 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0	7.886							0.17
10	8.869	0.146	8.149	0.151	7.319	0.14	6.738	0.153
20	10.549	0.121	10.238	0.128	8.922	0.136	7.52	0.145
30	11.262	0.117	10.154	0.120	9.124	0.131	8.847	0.135
40	12.438	0.165	11.394	0.179	10.652	0.163	9.185	0.188

Table 3.12 Triaxial shear test results for reinforced (compacted density of 0.984 g/cm³) pond ash samples at confining pressure of 1 kg/cm^2

Area ratio (%)	CONFINEMENT PRESSURE At 1 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0	5.948							0.194
10	7.112	0.131	6.963	0.137	6.86	0.14	5.697	0.123
20	8.772	0.126	8.416	0.129	7.954	0.138	7.916	0.145
30	9.023	0.121	8.817	0.126	8.772	0.131	8.621	0.125
40	9.237	0.117	9.162	0.121	9.048	0.134	8.872	0.139

Table 3.13 Triaxial shear test results for reinforced (compacted density of 0.984 g/cm³) pond ash samples at confining pressure of 0 kg/cm^2

Stone column dia	CONFINEMENT PRESSURE At 0 Kg/cm^2							
	Length ratio							
	1		0.75		0.5		0.25	
	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)	Stress (Kg/cm^2)	Strain (mm)
0 cm	0.167							0.057
2.2 Cm	0.214	0.127	0.181	0.132	0.174	0.137	0.159	0.144
2.6 Cm	0.168	0.119	0.154	0.124	0.141	0.127	0.136	0.135
3.5 Cm	0.152	0.095	0.142	0.117	0.129	0.124	0.119	0.129
4.0 Cm	0.132	0.081	0.123	0.092	0.104	0.117	0.081	0.125

Fig 3.3 Special equipments for cavity formation for installation of stone column

Fig 3.4 Constant Volume Mould with Arrangements for Imparting Compaction Energy

3.6.3 FOOTING LOAD TESTS

The test was conducted to find out the bearing capacity of pond ash beds, reinforced with stone column on a strain controlled loading machine. Compacted pond ash beds at density of 0.90 kg/cm³ and at OMC are prepared in the cylindrical tank size of internal diameter 396 mm and length of 400 mm. The stone column was inserted at the center of pond ash by hollow cylindrical steel tubes to give the required area ratio and to compact the stone aggregate iron rod was used to maintain the required density. The test was conducted at area ratio of 0, 10%, 20%, 30%, 40% and 60%. The length ratios of stone columns were also varied as 0, 0.25, 0.50, 0.75, and 1.0. A circular footing of 75mm placed centrally on the test bed was tested in a strain controlled loading machine with strain rate of 1.25 mm/minute till failure of the sample. The test result are presented in Table 3.15 and 3.16 and the sample preparation and instrument was used to make the sample were shown in Figs 3.4 to 3.7.

Fig 3.4 Hollow cylindrical pipe to make cavity on pond ash

Fig 3.5 Compacted pond ash

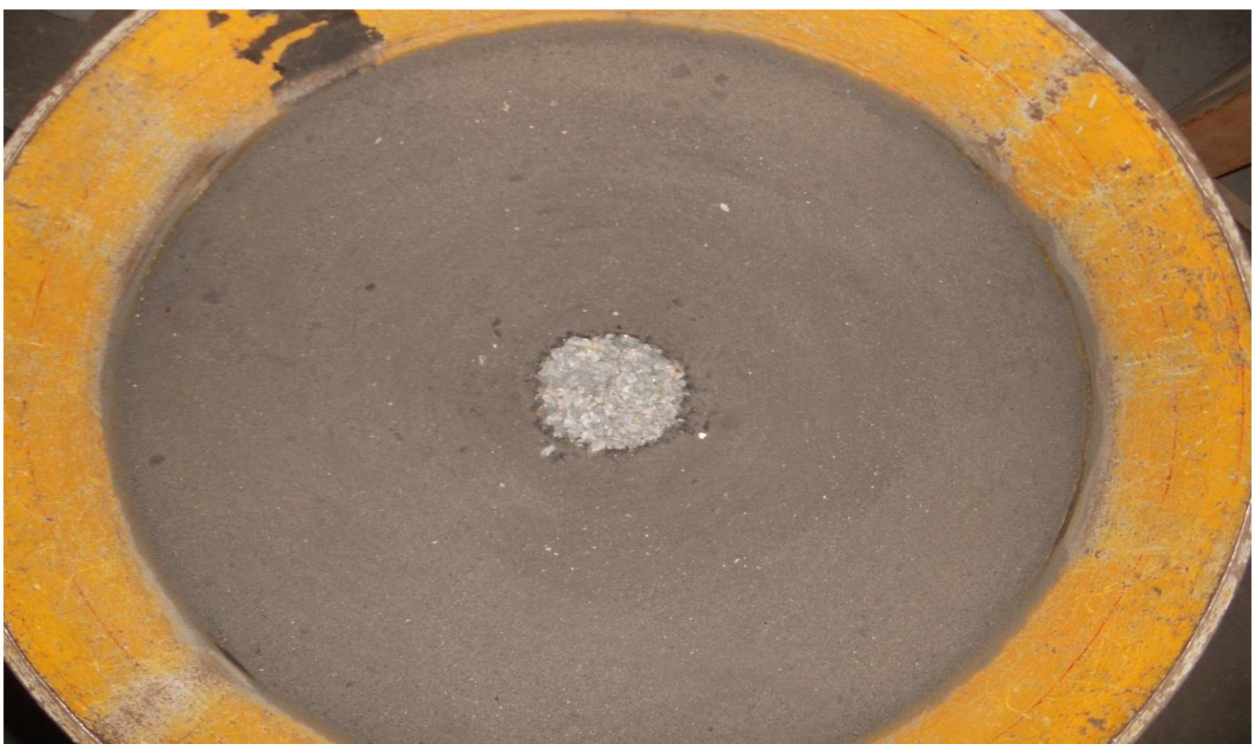


Fig 3.6 Reinforced pond ash by stone column

Fig 3.7 Footing load test

Table 3.14 Results of Footing Load Test

Length Ratio	Area Ratio (%)					
	0		10		20	
	Failure stress(kg/cm^2)	Strain (%)	Failure stress (kg/cm^2)	Strain (%)	Failure stress (kg/cm^2)	Strain (%)
1.0	2.675	11.33	4.124	19.3	4.868	25.33
0.75			3.754	24.	4.625	18
0.50			3.354	22.6	3.954	22.66
0.25			2.844	24.6	3.26	12
0			2.675	11.33	2.675	11.33

Table 3.14 (continued)

Length Ratio	Area Ratio					
	0		40		60	
	Failure stress (kg/cm^2)	Strain (%)	Failure stress(kg/cm^2)	Strain (%)	Failure stress(kg/cm^2)	Strain (%)
1.0	2.675	11.33	6.234	19.33	7.841	26.66
0.75			5.985	24.66	7.261	33.33
0.50			4.937	15.33	5.658	30.66
0.25			4.133	14	4.767	19.33
0			2.675	11.33	2.675	11.33

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

Fly ash is a by-product of the coal based thermal power plants contains grains of fine sand to silt size which makes acres of land unsuitable for all human purpose. Presently about 2000 acres of land are being covered by the ash pond. In the present work an attempt has been made to stabilize the deposits in ash pond by reinforcing it with stone columns and without stone column. The effect of replacement ratio, length of stone columns on the strength and stress strain behavior of composite columns has been evaluated. Further the effect of confining pressure and moulding pressure on the strength of composite columns has also been investigated. Stabilization of these abandoned ash ponds using stone. In present work the strength change behavior of reinforced pond ash samples with different parameters of stone column are evaluated by confining pressure and unconfined pressure and also test has conducted of pond ash inside the cylindrical tank with different area ratio and length ratio of stone column reinforced to pond ash. Here two set of test series has conducted one is without reinforcing stone column and another with reinforcing of stone column. The detail tests results are presented and discussed in this chapter.

4.2 TEST SERIES-1

4.2.1 INDEX PROPERTIES

4.2.1.1 Specific Gravity

Specific gravity is one of the important physical properties needed for the use of coal ashes for geotechnical and other applications. In general, the specific gravity of coal ashes lies around 2.0 but can vary to a large extent (1.6 to 3.1). The variation of specific gravity of the coal ash is the result of a combination of many factors such as gradation, particle shape and chemical composition. The reason for a low specific gravity could either be due to the presence of large number of hollow cenospheres from which the entrapped micro bubbles of air cannot be removed, or the variation in the chemical composition, in particular iron content, or both .The specific gravity of pond ash was determined according to IS: 2720 (Part-III) -1980 guidelines by pycnometer method with water. The average specific gravity value found to be 2.30. The specific gravity of pond ash was found to be lower than that of the conventional earth material.

4.2.1.2 Determination of grain size

The grain size distribution curve of pond ash is presented in Fig 4.1.The pond ash consists of grains mostly of fine sand to silt size. The coefficient of uniformity and coefficient of curvature of pond ash sample is found to be 6.13 and 2.61 respectively indicating uniform gradation of sample. The grain size distribution of pond ash mostly depends upon the degree of pulverization of coal and the firing temperature in boiling units. This modern plant having more efficient coal pulverizing equipment tends to produce ashes of finer texture than those from older stations. As the present pond ash sample is from the ash pond of R.S.P, the presence of sediment foreign particles are also expected to present in it. Atterberg Limits was not possible to find out the liquid limit and plastic limit of pond ash indicating that pond ash is non-plastic in nature.

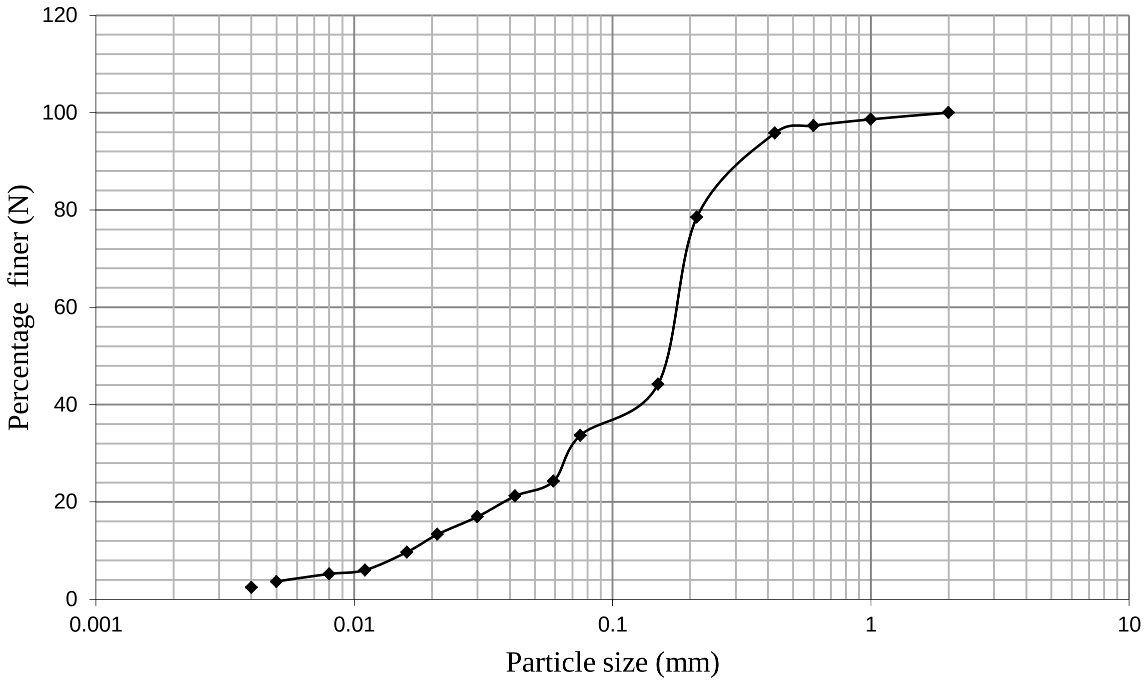


Fig 4.1: Grain size distribution curve

4.2.2 ENGINEERING PROPERTIES

4.2.2.1 Compaction Characteristics

The compaction characteristics of pond ash with different compaction energies have been studied by varying the compaction energies 119, 357, 595, 1604 and 2674Kj/m³ of compacted volume. The OMC and MDD of pond ash samples corresponding to these compactive efforts have been evaluated and presented in Table. Relationship between dry density and moisture content of pond ash at different compaction energies have been shown in Fig. It is seen that as the compactive energy increases the MDD increases and the water required to achieve this density is reduced. A continuous increase in the value of MDD is observed with the compactive energy (Fig.4.3). Plot between OMC and compactive energy (Fig.4.4) shows that initially the OMC decreases rapidly with compactive effort and then the rate of decrease is not that prominent .The MDD of specimens is found to change from 0.984 to 1.23 gm/cm³ with change in compaction energy from 119 to 2674kJ/m3 whereas the OMC is found to decrease from 43.23 to 31.7%. This shows that the compacted density of pond ash responds very poorly to the compaction energy. This may be attributed to the rounded shape of particles and uniform gradation of the sample. There are many factors like gradation, carbon content, iron content and fineness etc., mainly control the compaction characteristics of pond ash.

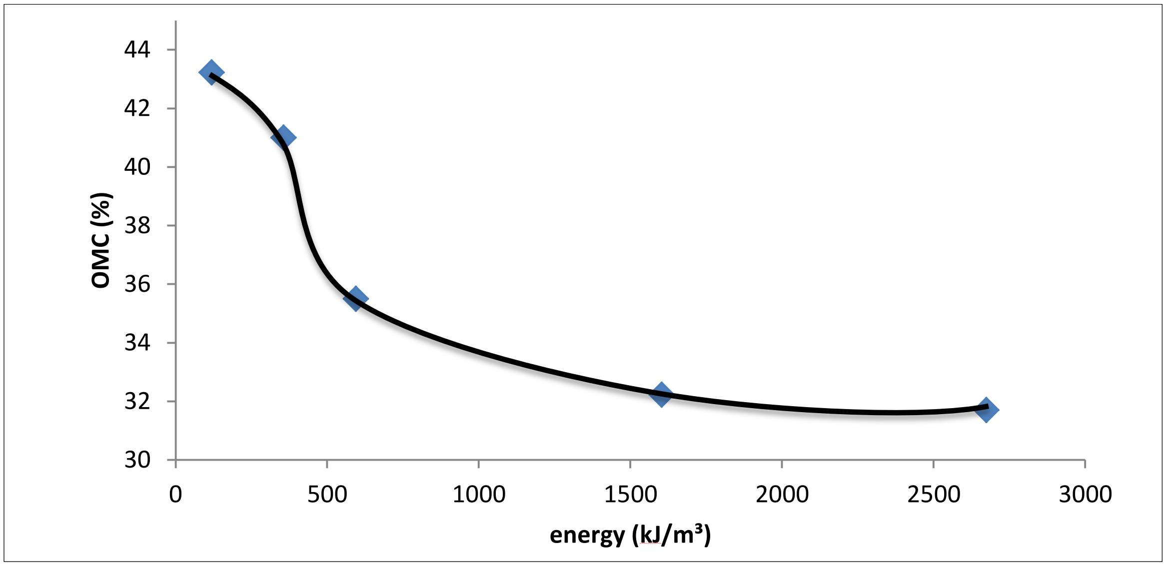


Fig 4.2: Variation of OMC at different compactive level

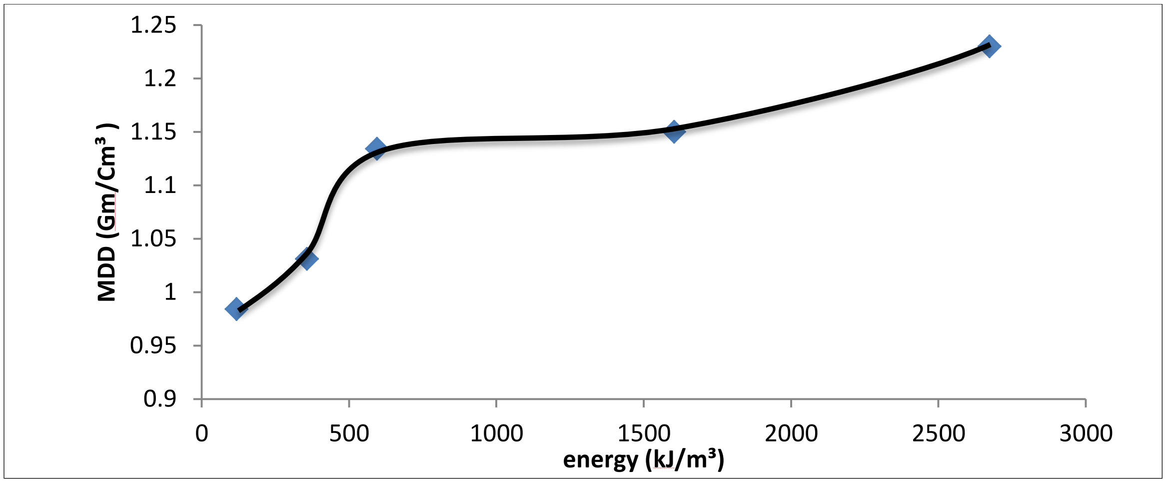
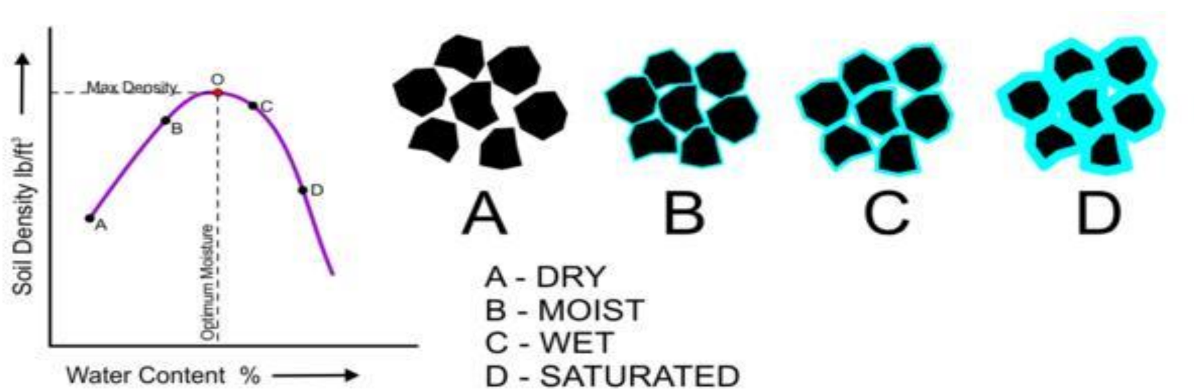


Fig 4.3: Variation of MDD at different compactive level

4.2.2.2 Effect of saturation on shear parameter

It was found that from direct shear test as the increase of compaction energy the dry density, angle of internal friction also increasing gradually. However the OMC decreases drastically with increase of compaction energy. When the same sample was conducted on direct shear test at saturation same thing has happen as OMC of respective compaction energy and dry density, angle of internal friction also increasing gradually. However the OMC decreases drastically with increase of compaction energy. From the both case at OMC and saturation which result has got at saturation dry density and angle of internal friction is less than OMC result. Cohesion value of pond ash has increased due to addition of water and compaction energy, due to compaction energy the particle get come closer, the pond ash has some surface activity due to which cohesion value has increased. On the case of saturation the particle has lose its strength of surface activity and cohesion value has decreased as compare to OMC. Angle of internal friction basically depends upon compaction energy it will show maximum at OMC, due to the maximum compaction energy on the case of saturation angle of internal friction has decreased due to water particle will behave as a lubricate effect on the surface of ash pond particle.



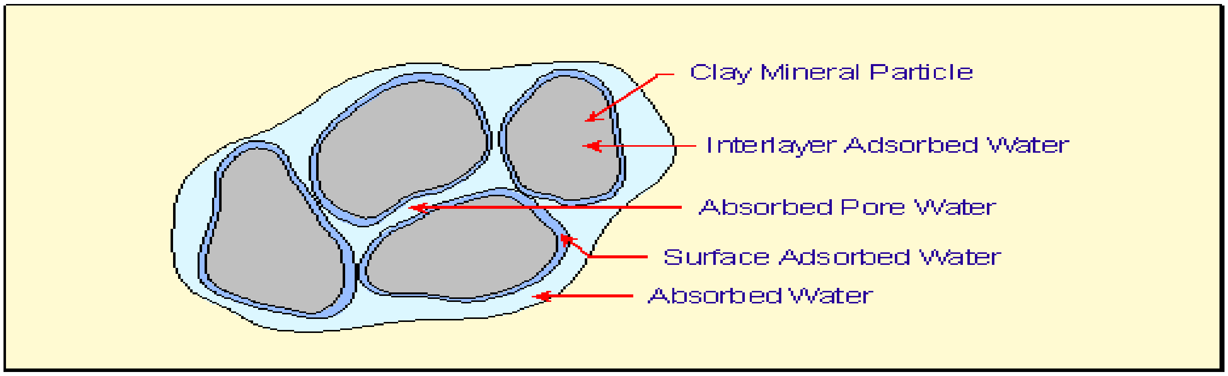


Fig 4.4: Absorbed and adsorbed water in clay-water systems

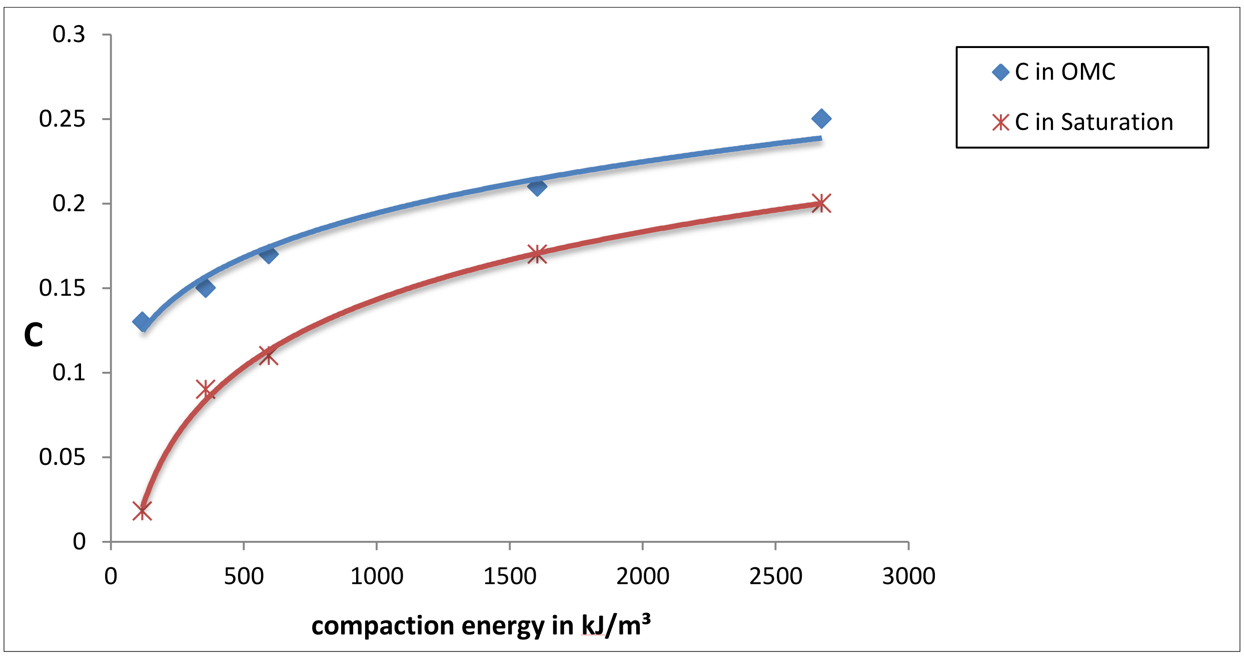


Fig 4.5: Variation of unit cohesion at OMC and saturation under different compactive level

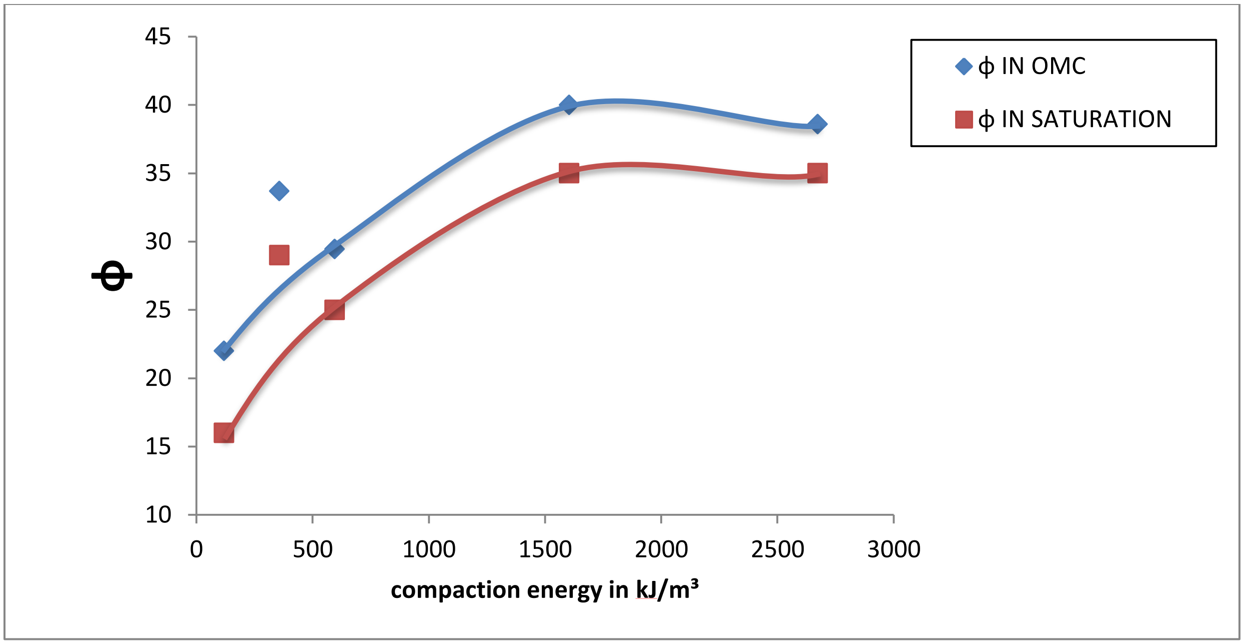


Fig 4.6: Variation of frictional angle at OMC and saturation under different compactive level

4.2.2.3 Determination of Unconfined Compressive Strength

4.2.2.3.1 Effect of Compaction Energy at OMC

Unconfined compressive strength tests were carried out on untreated pond ash specimens compacted to their corresponding MDD at OMC with compactive effort varying as 119, 357, 595, 1604 and 2674Kj/m^3. The stress-strain relationships of compacted pond ash were presented in Fig-4.9 Form these plots it is observed that the failure stress as well as initial stiffness of samples, compacted with greater compaction energy, are higher than the samples compacted with lower compaction energy. The immediate compressive strength of pond ash is 19.587 kPa at compaction energy of 119 kJ/m^3 which increase to 66.758 kPa at compaction energy of 2674 kJ/m^3.However in general the failure strains are found to be lower for samples compacted with higher energies. The failure strains vary from a value of 2.25 to 2.75%, indicating brittle failures in the specimens at sample prepared on higher density shown in Fig- and showing bulging failure under lower density sample shown in Fig-. The increase in unconfined strength and initial stiffness of specimens with increased compactive effort is attributed to the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component in the sample.

Fig-4.7:Bulging failure of compacted pond ash

Fig 4.8:Cracking failure of compacted pond ash

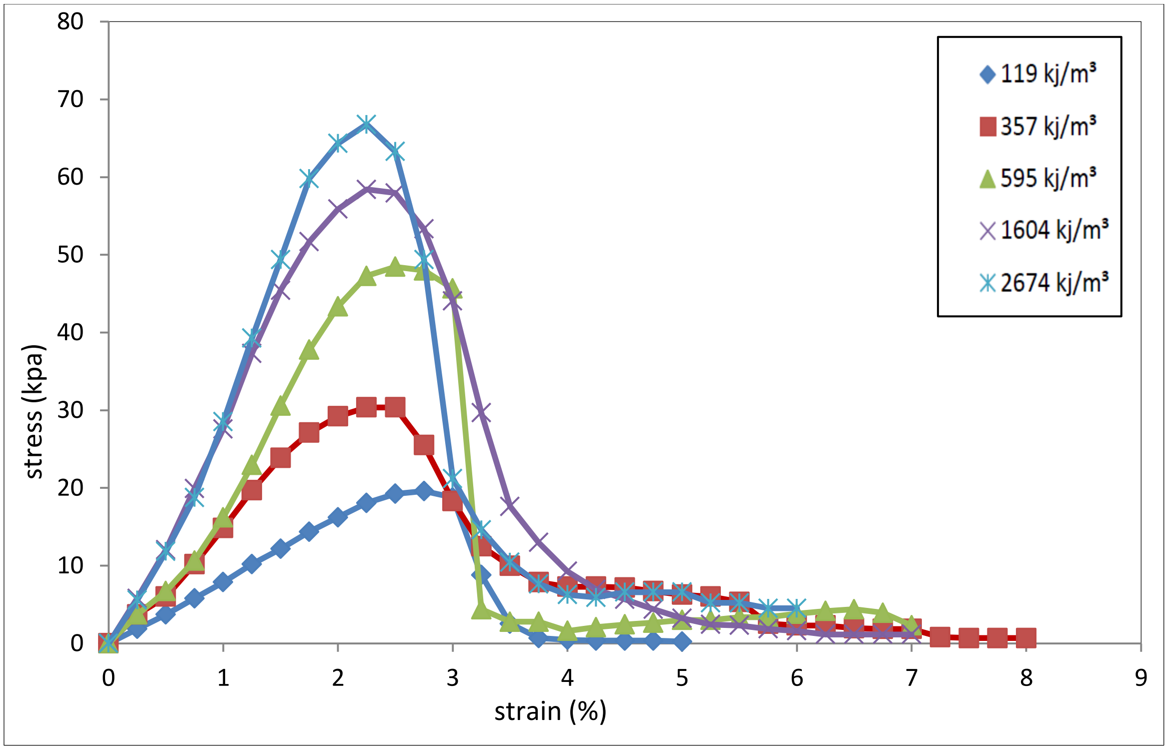


Fig 4.9: variation of failure stress-strain in different compactive energy

4.2.2.3.2 Effect of Compaction Energy at saturation

Unconfined compressive strength tests were carried out at saturation on untreated pond ash specimens compacted to their corresponding MDD at OMC with compactive effort varying as 119, 357, 595, 1604 and 2674Kj/m³ then the sample were covered with wax to saturate the sample. The samples were keeping for 30 minute for proper saturation. The stress-strain relationships of compacted saturated pond ash were presented in Fig-4.11. Form these plots it is observed that the failure stress as well as initial stiffness of samples, compacted with greater compaction energy, are higher than the samples compacted with lower compaction energy. The immediate compressive strength of pond ash is 8.142 kPa at compaction energy of 119 kJ/m³ which increase to 38.45 kPa at compaction energy of 2674 kJ/m³.However in general the failure strains are found to be lower for samples compacted with higher energies.

The failure strains vary from a value of 2.5 to 3%, indicating brittle failures on the both specimens at sample prepared on higher density and lower density sample. Due to the saturation of the sample interlocking between pond ash particle and the void space has filled with small size pond ash particle with some quantity of water and it was proper dense as compare to sample prepare at OMC. So that the stress at saturation is higher as compare to OMC. The increase in unconfined strength and initial stiffness of specimens with increased compactive effort is attributed to the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component in the sample.

Fig 4.10: cracking failure of saturated pond ash covered with wax

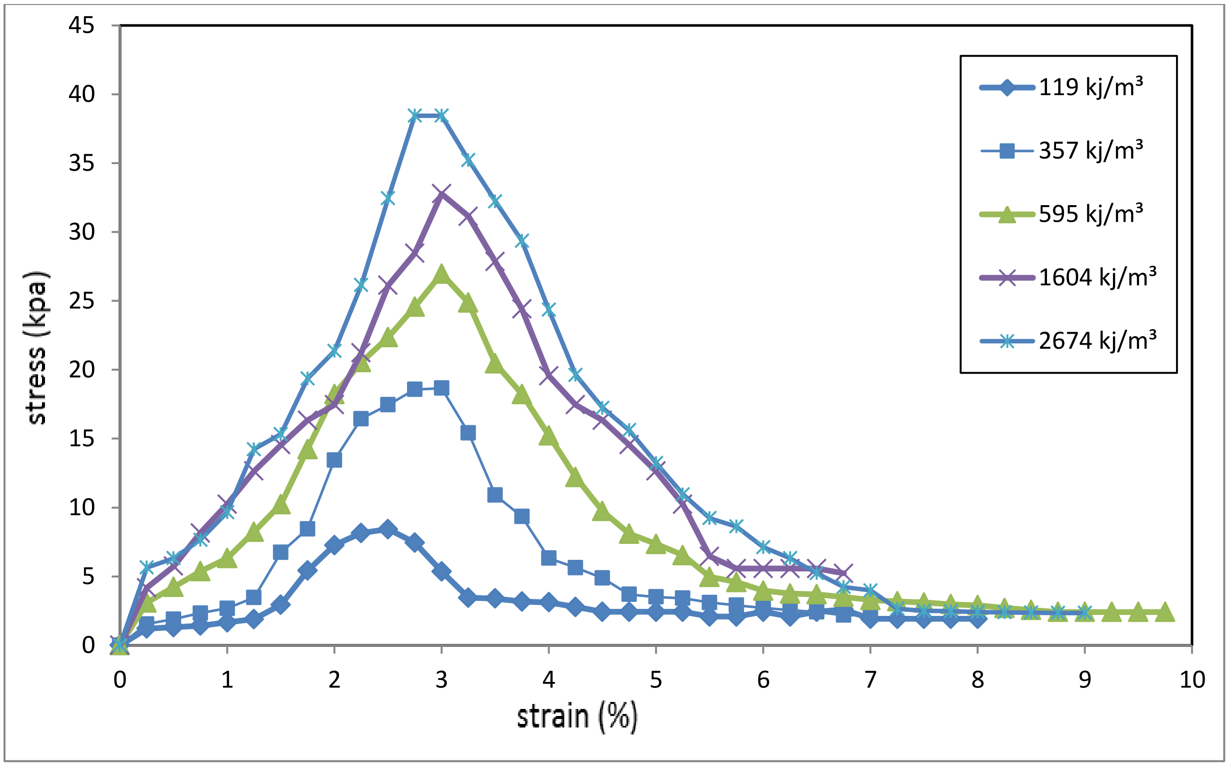


Fig 4.11: variation of failure stress-strain in different compactive energy

4.2.2.4 Determination of confined Compressive Strength of pond ash by Traiaxial test

Triaxial tests were carried out on untreated pond ash specimens compacted to their corresponding MDD at OMC with compactive effort varying as 119, 357, 595, 1604 and 2674Kj/m³.That sample were prepared in dimension of 50mm(dia)x100mm(height) on five respective density of their corresponding compaction energy, on each density to study the effect of confining pressure there were given three confinement pressure was applied as 1,2,3Kg/cm² .The relation between strain and stress was plotted in Fig-4.11,4.12,4.13,4.14 and 4.15 .From these plot it is observed that under all the confining pressure to their respective compaction energy, the stress value was increasing with the increase of confining pressure from 1kg/cm^2 to 3kg/cm^2 due to the confinement. The stress value was increased from 3.05kg/cm^2 to 8.15 kg/cm^2 to their respective increase of confining pressure from

$1kg/cm^2$ to $3kg/cm^2$ in compactive energy $2674kJ/m^3$. The stress value was increased from $2.63kg/cm^2$ to 7.17 kg/cm^2 to their respective increase of confining pressure from $1kg/cm^2$ to $3kg/cm^2$ in compactive energy $1604kJ/m^3$. The stress value was increased from $2.02kg/cm^2$ to 5.94 kg/cm^2 to their respective increase of confining pressure from $1kg/cm^2$ to $3kg/cm^2$ in compactive energy $595kJ/m^3$. The stress value was increased from $2.02kg/cm^2$ to 5.87 kg/cm^2 to their respective increase of confining pressure from $1kg/cm^2$ to $3kg/cm^2$ in compactive energy $357kJ/m^3$. The stress value was increased from $1.67kg/cm^2$ to 4.7 kg/cm^2 to their respective increase of confining pressure from $1kg/cm^2$ to $3kg/cm^2$ in compactive energy $119kJ/m^3$. The failure stress of $1kg/cm^2$ not sufficient to make the sample failure at $3kg/cm^2$, due to the confinement and sample prepared at higher compactive effort attributed to the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component and angle of internal friction in the sample.so that the unit cohesion was increased from 0.106 kg/cm^2 to 0.239 kg/cm^2 and angle of internal friction was increased from 19.87^0 to 37.4^0.

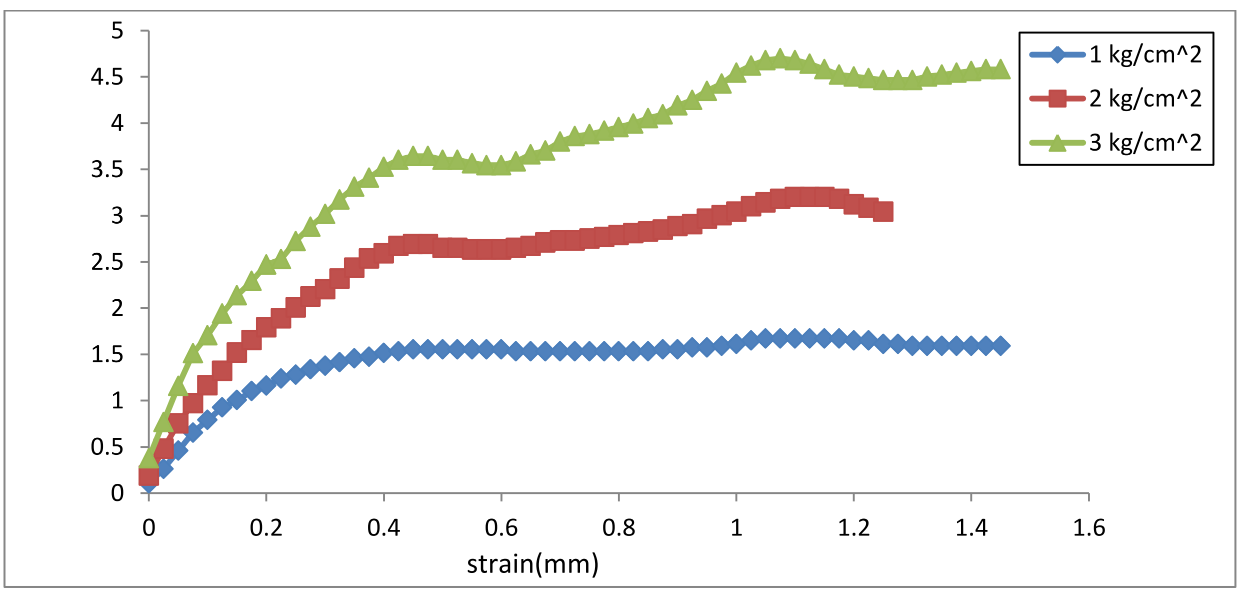


Fig 4.12: Sample prepared on compaction energy 119 kJ/m^3

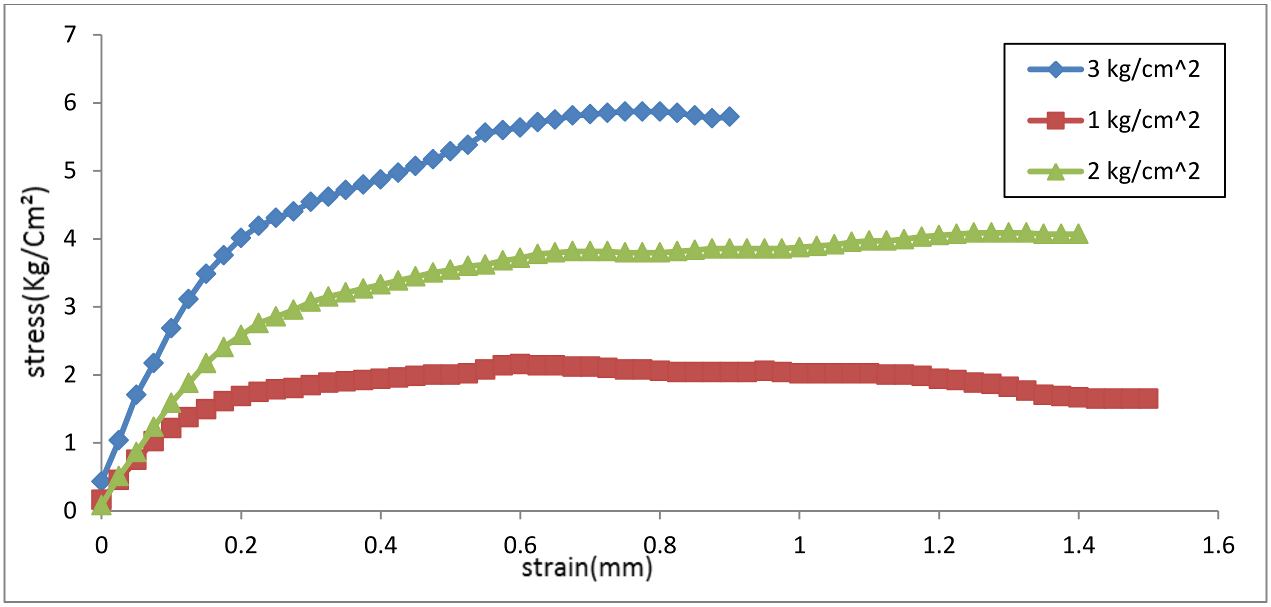


Fig 4.13: Sample prepared on compaction energy 357 kJ/m³

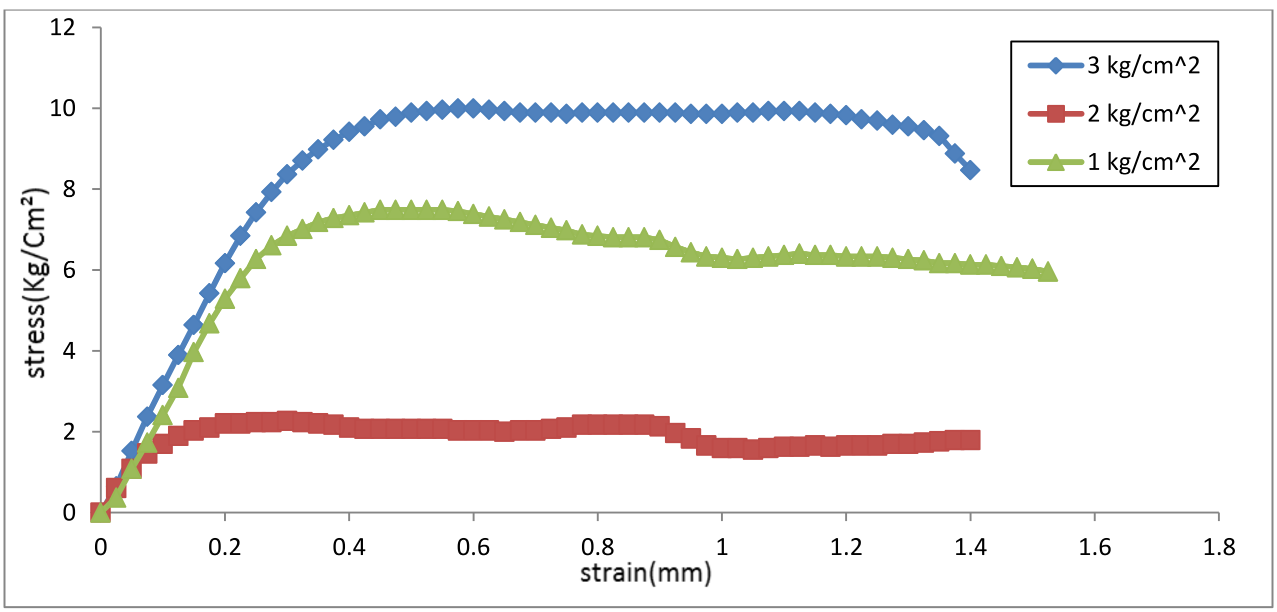


Fig 4.14: Sample prepared on compaction energy 595 kJ/m³

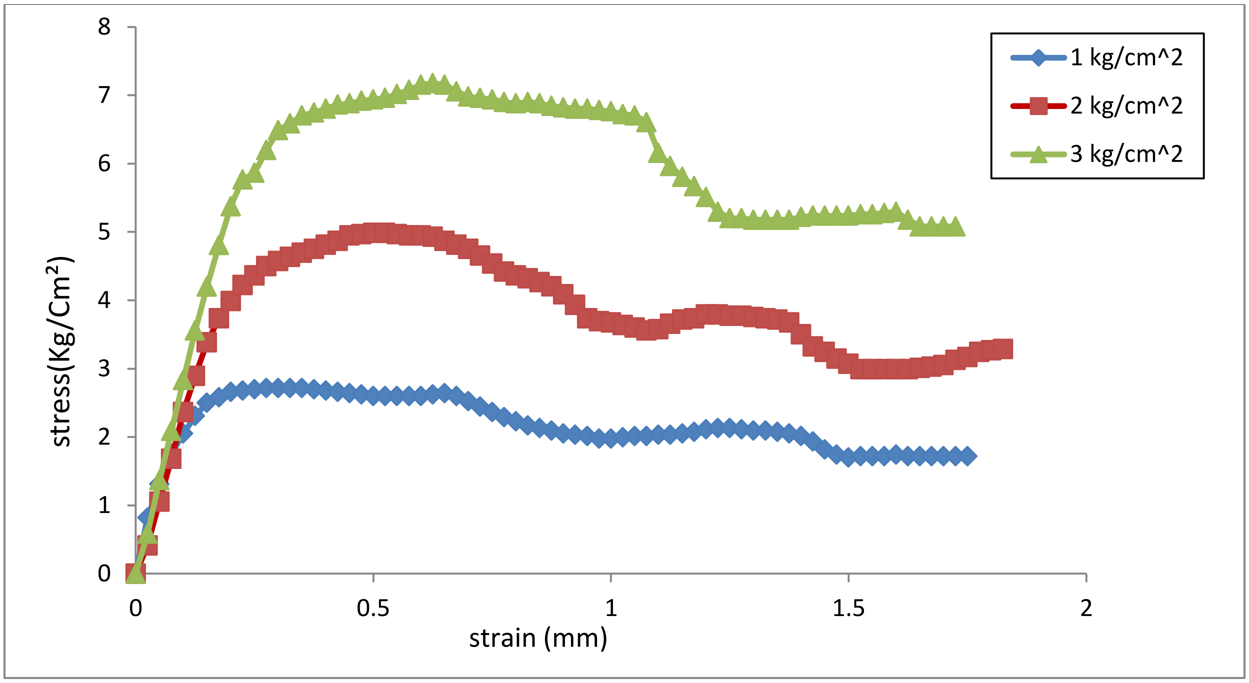


Fig 4.15: Sample prepared on compaction energy 1604 kJ/m³

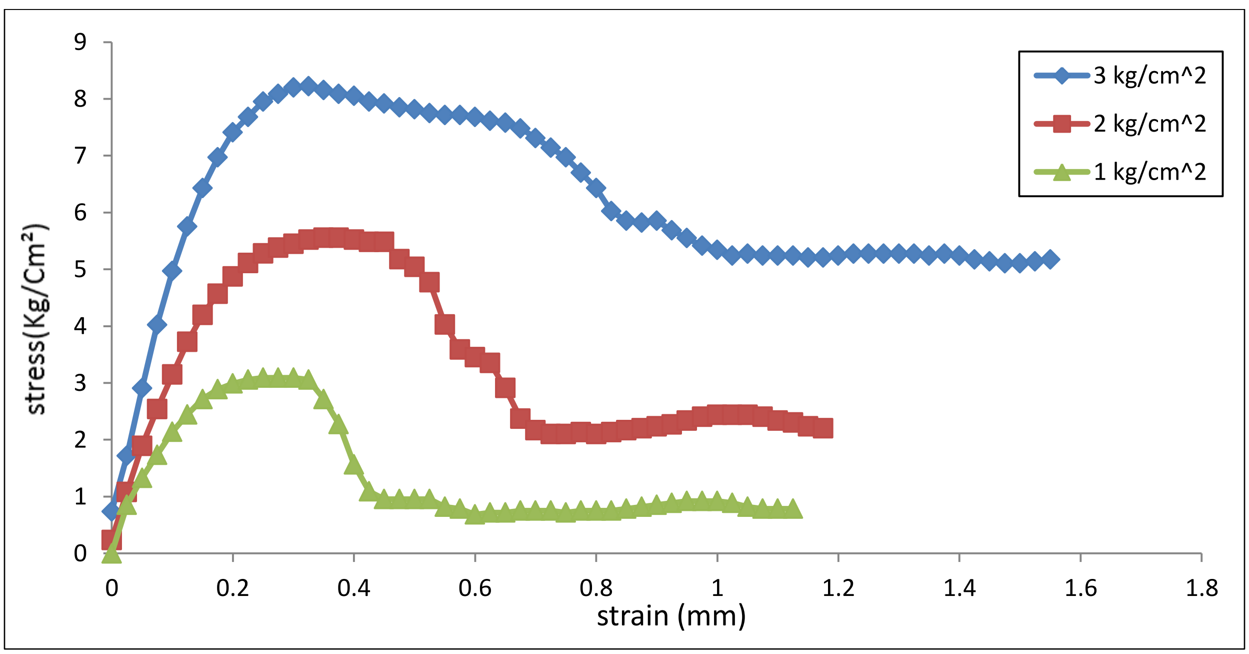


Fig 4.16: Sample prepared on compaction energy 2674 kJ/m³

4.3 Test series-2

4.3.1 Determination of Unconfined Compressive Strength of pond ash reinforced with stone column

Experimental work was done on the reinforced pond ash by stone column to study the behavior under varying radius of stone column to maintain the area ratio of 10%, 20%, 30% and 40% and along with to study the effect of length ratio on pond ash by varying the length of stone column to provide the length ratio of 0.25Lr, 0.50 Lr, 0.75 Lr, and 1.0 Lr.The stress strain response of reinforced stone column under different condition has study briefly here and the failure pattern of reinforced pond ash has shown in Fig-4.17 and 4.18.

It is found that reinforced of pond ash by 10% area ratio and by varying its length ratio it has observed that the sequence of stress were as full length stone column Lr was showing maximum stress then 0.75Lr length stone column then 0. 5Lr and then 0.25 Lr length stone columns then without reinforced stone column. Stress values were increasing by increasing the length ratio of stone column. The results were shown in Fig-4.19.

At 20% area ratio stone column with the varying in length ratio by 1Lr,0.75Lr,0.5Lr and 0.25Lr the sequence of stress were as 0.75 Lr was showing maximum stress then 0.5Lr length stone column then without reinforced stone column then 0. 25Lr and then 1 Lr length stone column. The results were shown in Fig-4.20.

At 30% area ratio stone column with the varying in length ratio by 1Lr,0.75Lr,0.5Lr and 0.25Lr the sequence of stress were as 0.25 Lr was showing maximum stress then without reinforced stone column then 0.5Lr length stone column then 0. 75Lr and then 1 Lr length stone column. The results were shown in Fig-4.21.

At 40% area ratio stone column with the varying in length ratio by 1Lr,0.75Lr,0.5Lr and 0.25Lr the sequence of stress were as without reinforced stone column then was showing maximum stress then 0.25 Lr length stone column then 0.5Lr length stone column then 0. 75Lr and then 1 Lr length stone column. The results were shown in Fig-4.22.

From the above experimental analysis it show that as increasing of area ratio of reinforced pond ash the stress value has decreased with the decreased of strain. At area ratio 10% it has observed that stress value was increasing by increase order of length ratio of stone column whereas at 40% area ratio it was showing reverse. It was due to adequate amount of confining pressure was not sufficient to keep stable sample prepare at 40% area ratio. At 20% and 30% area ratio there is some variation on sequence of stress by the sequence of length ratio.

Fig 4.17: side view of reinforced pond ash cracking failure

Fig 4.18: top view of reinforced pond ash cracking failure

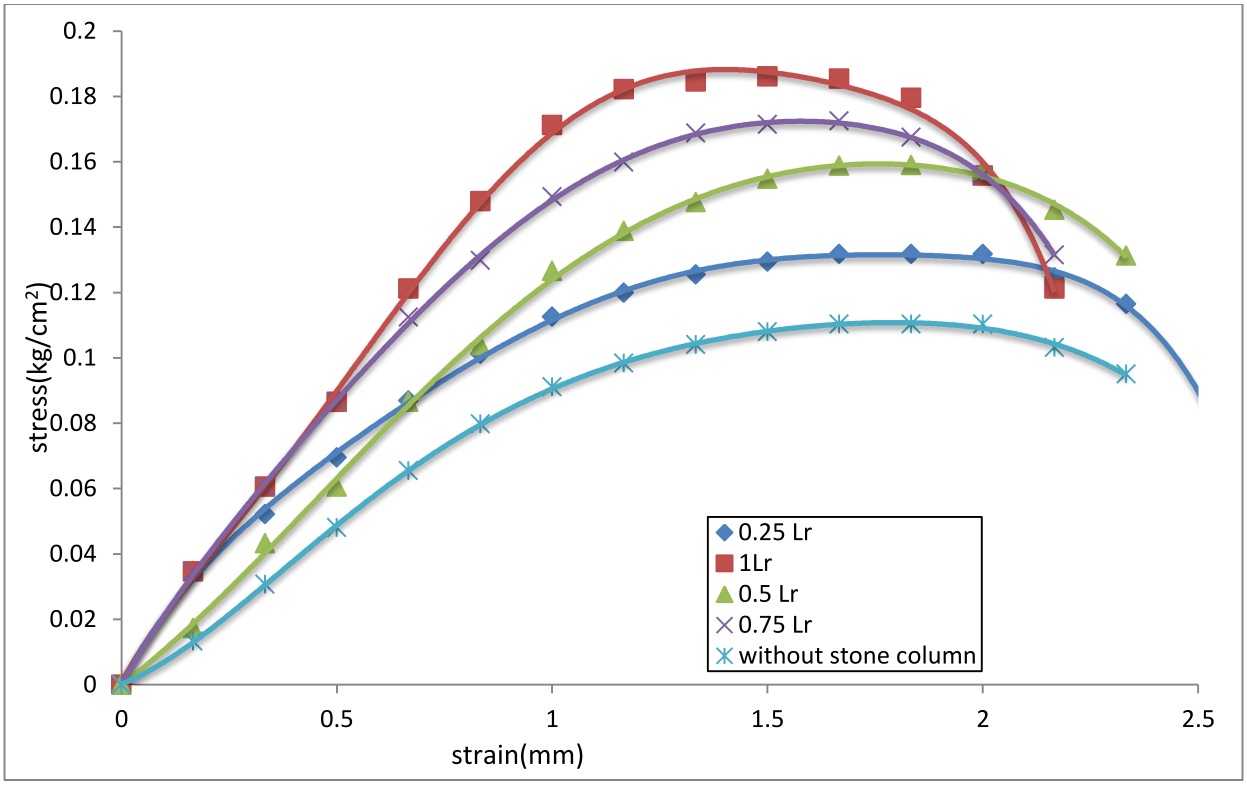


Fig 4.19: pond ash reinforced with 2.2cm dia stone column

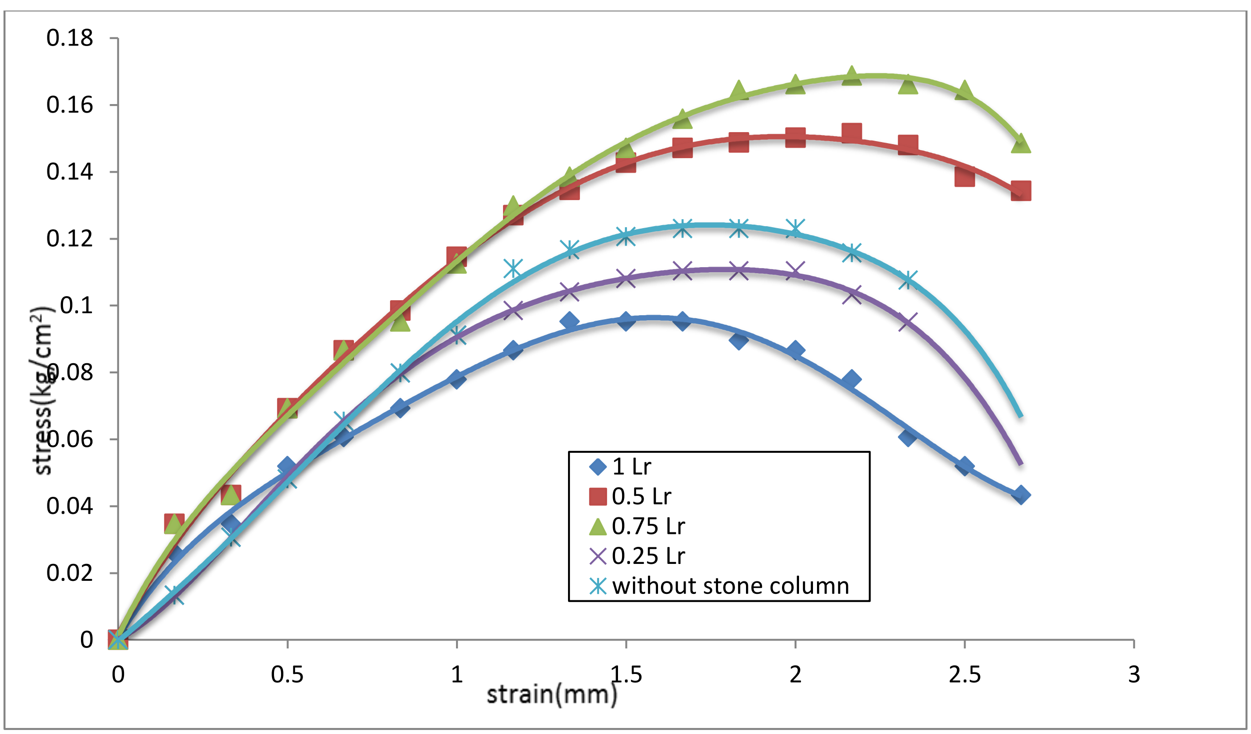


Fig 4.20: pond ash reinforced with 2.6cm dia stone column

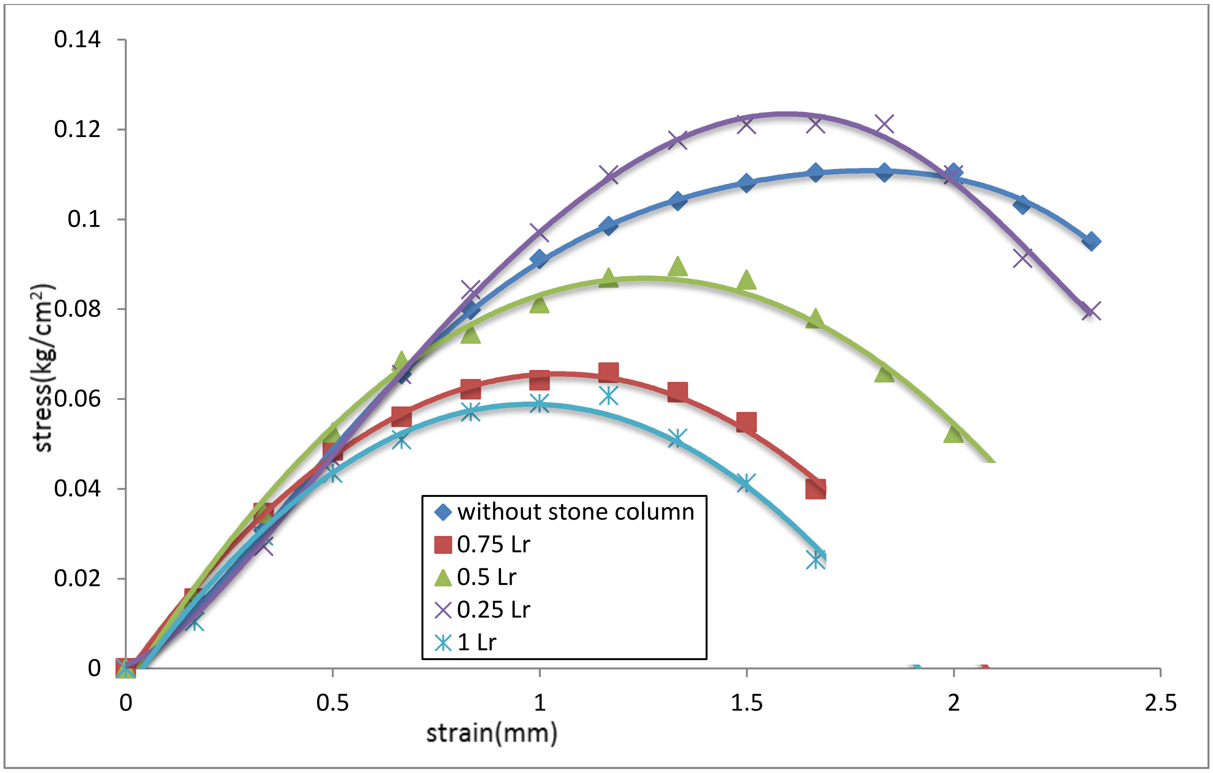


Fig 4.21: pond ash reinforced with 3.5cm dia stone column

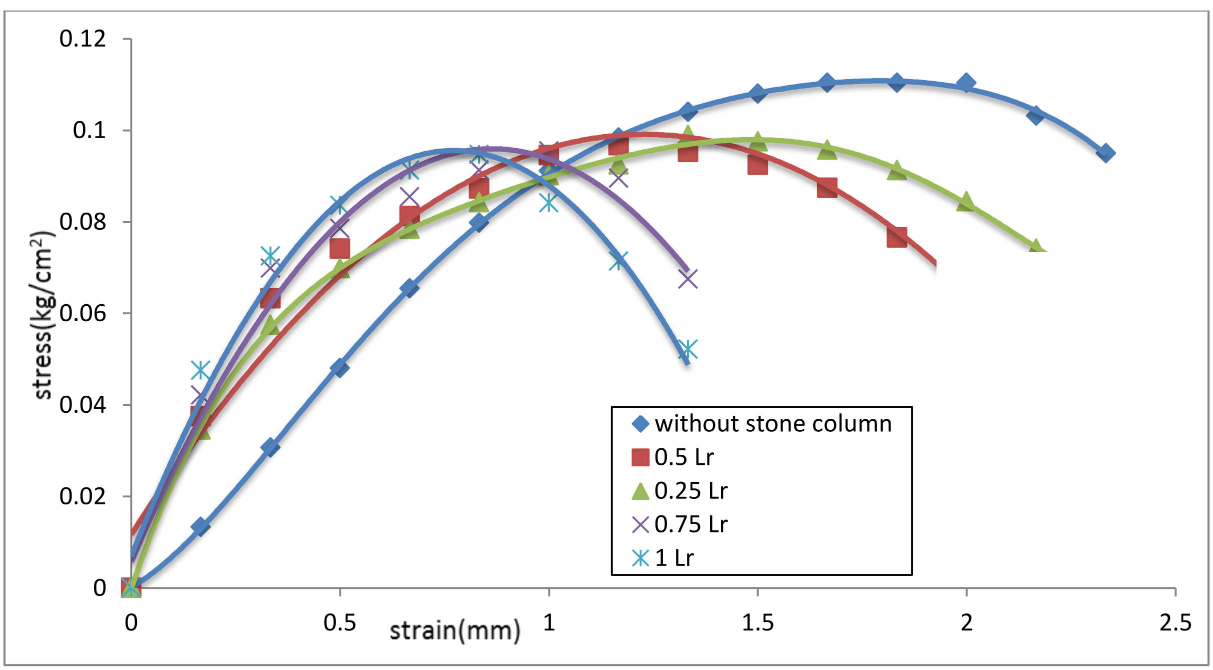


Fig 4.22: pond ash reinforced with 4cm dia stone column

4.3.2 Determination of Triaxial test of pond ash reinforced with stone column

Triaxial tests were carried out on untreated pond ash specimens compacted to their corresponding MDD at OMC with compactive effort of 94kJ/m^3.Here the test was conducted to study the response of pond ash by varying area ratio along with the effect of of their corresponding length ratio .That sample were prepared in dimension of 75mm(dia)x150mm(height) respective density of their corresponding compaction energy, on each density to study the effect of confining pressure there were given three confinement pressure was applied as 1,2,3Kg/cm² .The relation between strain and stress was plotted in Fig-4.23,4.24,4.25,4.26,4.27,4.28,4.29 and 4.30.

From these table and graph it has observed that under the confining pressure of 3Kg/cm² in 10% area ratio the stress value was increased from 9.41Kg/cm² to 10.89Kg/cm² by the increase of length ratio from 0.25 to 1 ,when compare with without reinforced stone column, without reinforced stone column shown maximum stress from 0.25 reinforced length ratio. Under the confining pressure of 2Kg/cm² in 10% area ratio the stress value was increased from 6.12Kg/cm² to 7.86Kg/cm² by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown maximum stress from 0.25 reinforced length ratio. Under the confining pressure of 1Kg/cm² in 10% area ratio the stress value was increased from 3.48Kg/cm² to 4.08Kg/cm² by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown maximum stress from 0. 5 reinforced length ratio.

Under the confining pressure of 3Kg/cm² in 20% area ratio the stress value was increased from 10.25Kg/cm² to 12.87Kg/cm² by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone columns. Under the confining pressure of $2Kg/cm^2$ in 20% area ratio the stress value was increased from $6.94Kg/cm^2$ to $9.149Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone column. Under the confining pressure of $1Kg/cm^2$ in 20% area ratio the stress value was increased from $3.89Kg/cm^2$ to $4.85Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone column.

Under the confining pressure of $3Kg/cm^2$ in 30% area ratio the stress value was increased from $11.85Kg/cm^2$ to $14.21Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone columns. Under the confining pressure of $2Kg/cm^2$ in 30% area ratio the stress value was increased from $7.89Kg/cm^2$ to $10.24Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone columns. Under the confining pressure of $1Kg/cm^2$ in 30% area ratio the stress value was increased from $4.24Kg/cm^2$ to $5.68Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone column.

Under the confining pressure of $3Kg/cm^2$ in 40% area ratio the stress value was increased from $12.89Kg/cm^2$ to $15.87Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone columns. Under the confining pressure of $2Kg/cm^2$ in 40% area ratio the stress value was increased from $8.985Kg/cm^2$ to $11.25Kg/cm^2$ by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone columns. Under the confining pressure of 1Kg/cm² in 40% area ratio the stress value was increased from 4.94Kg/cm² to 6.08Kg/cm² by the increase of length ratio from 0.25 to 1 , when compare with without reinforced stone column, without reinforced stone column shown low stress from other reinforced stone column.

Along with when compare the area ratio of their respected length ratio with other confining pressure the stress value was increased by increase of confining pressure. So here due to full length of stone column and confining pressure the stone column show more effective as compare to other because of the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component and angle of internal friction in the sample.

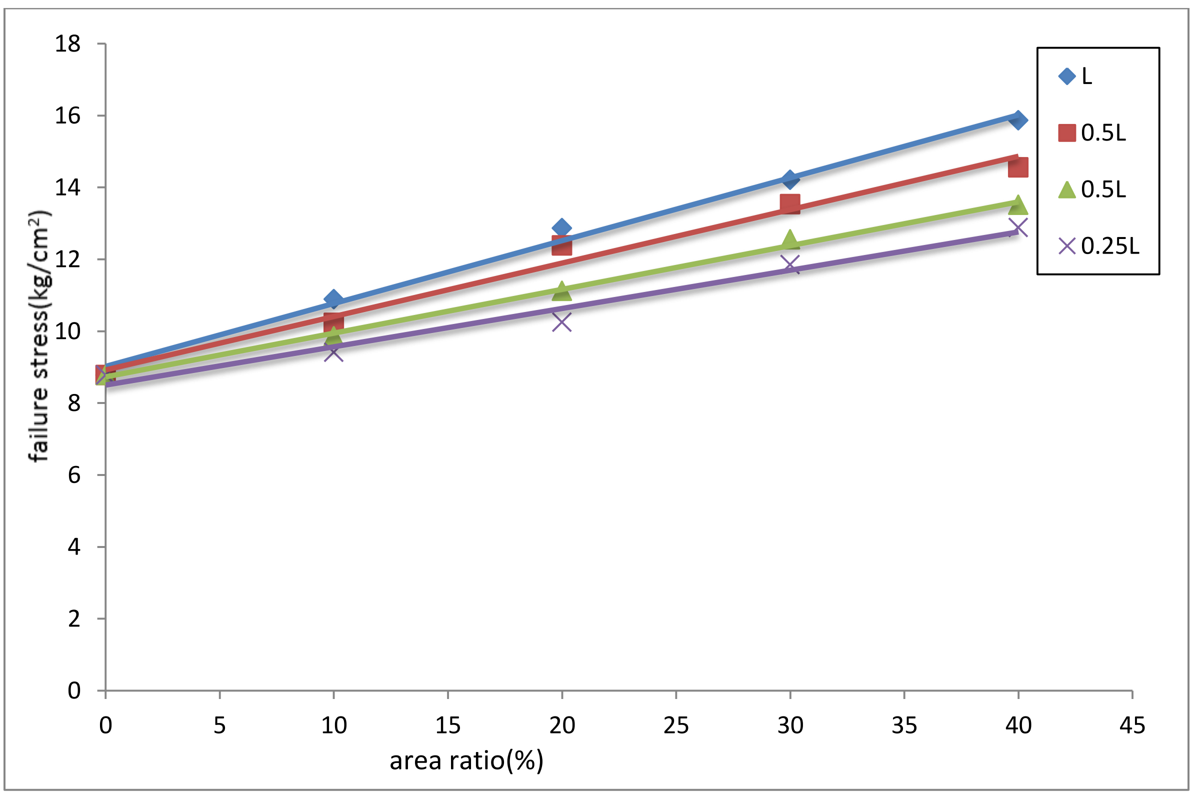


Fig 4.23:variation of failure stress with area ratio at 3kg/cm^2 confinement

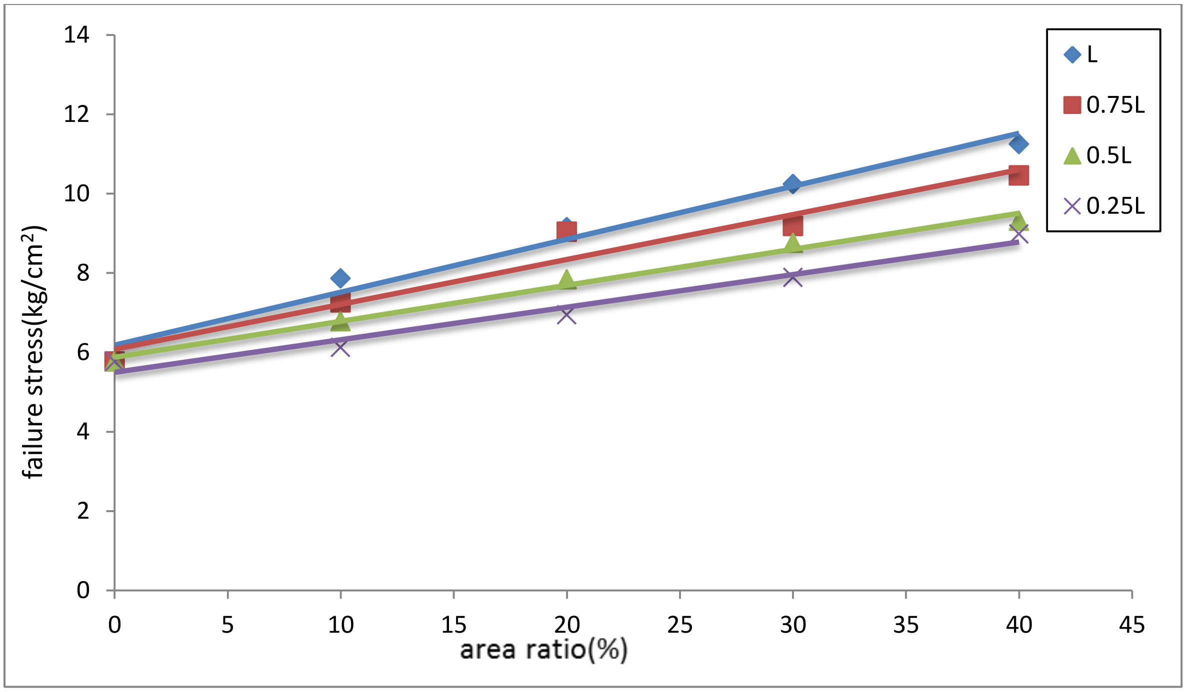


Fig 4.24: variation of failure stress with area ratio at 2kg/cm^2 confinement

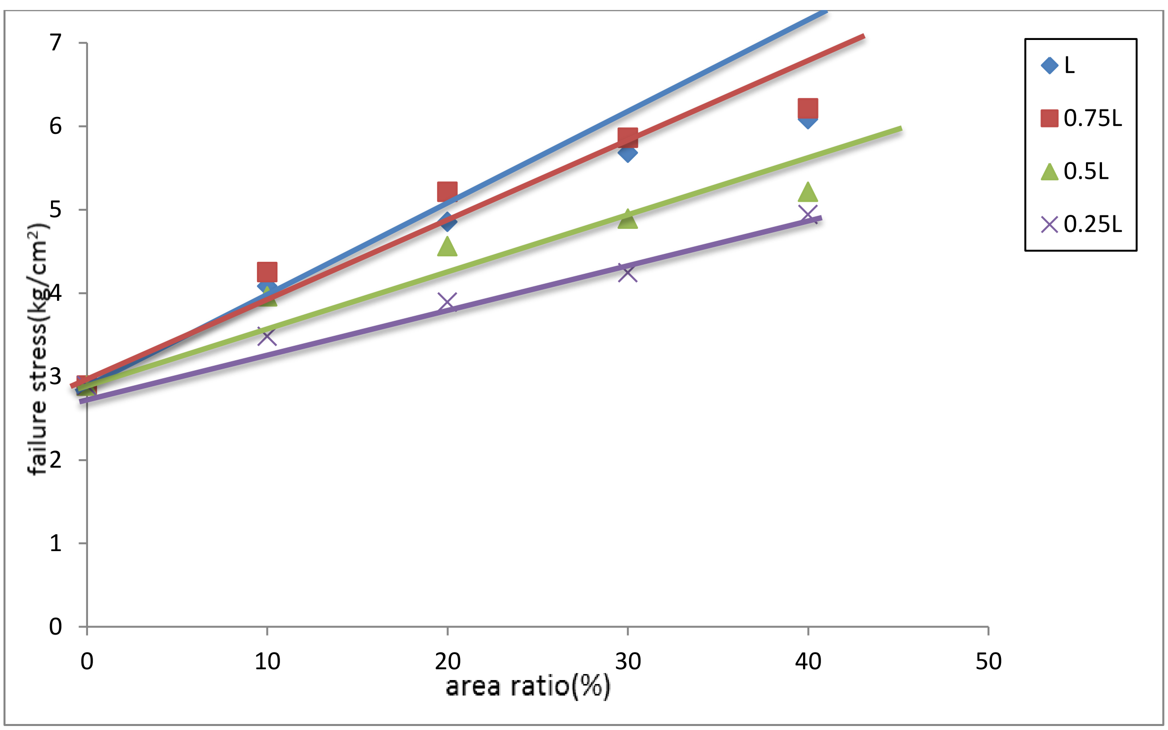


Fig 4.25: variation of failure stress with area ratio at 1kg/cm^2 confinement

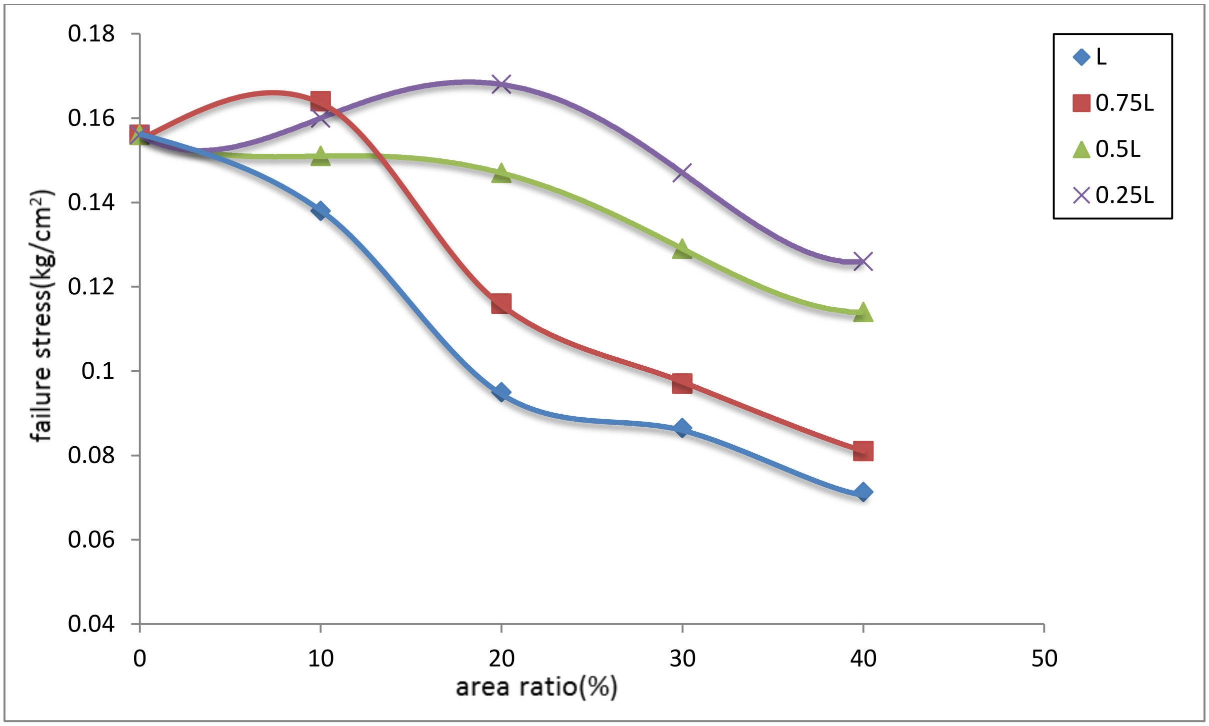


Fig 4.26: variation of failure stress with area ratio at 0kg/cm^2 confinement

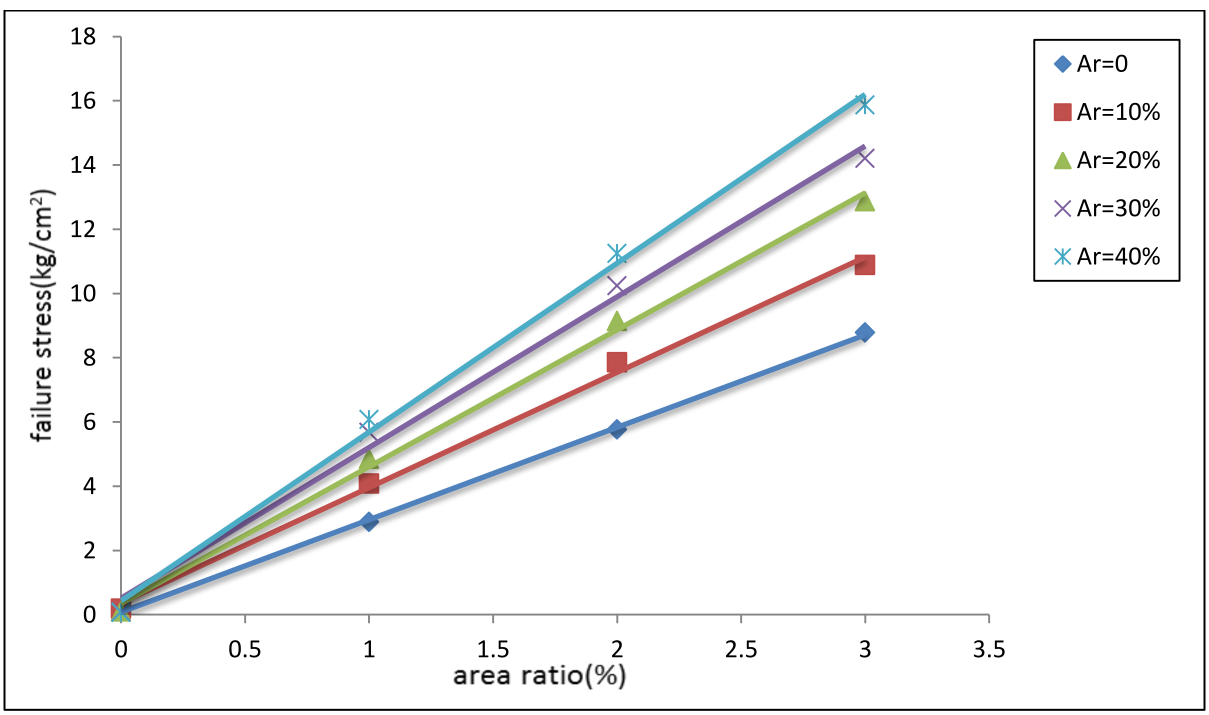


Fig 4.27: variation of failure stress in Full length reinforced pond ash in different confinement pressure

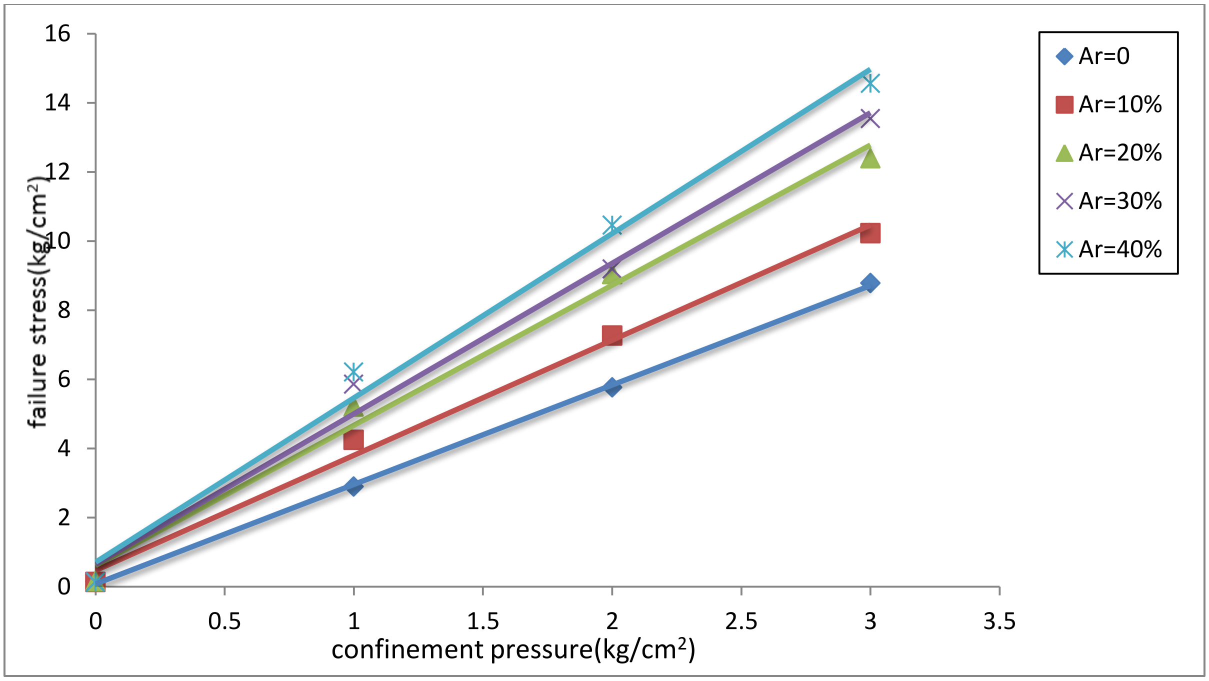


Fig 4.28: variation of failure stress in 0.75 length reinforced pond ash in different confinement pressure

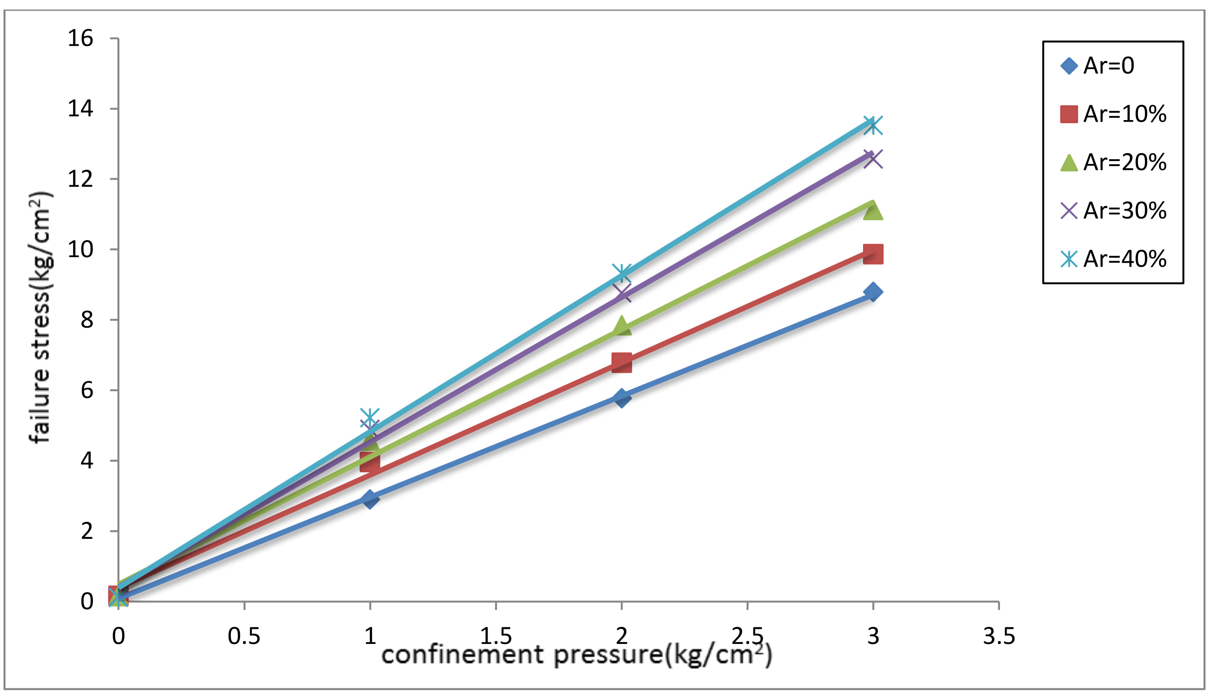


Fig 4.29: variation of failure stress in 0.5 length reinforced pond ash in different confinement pressure

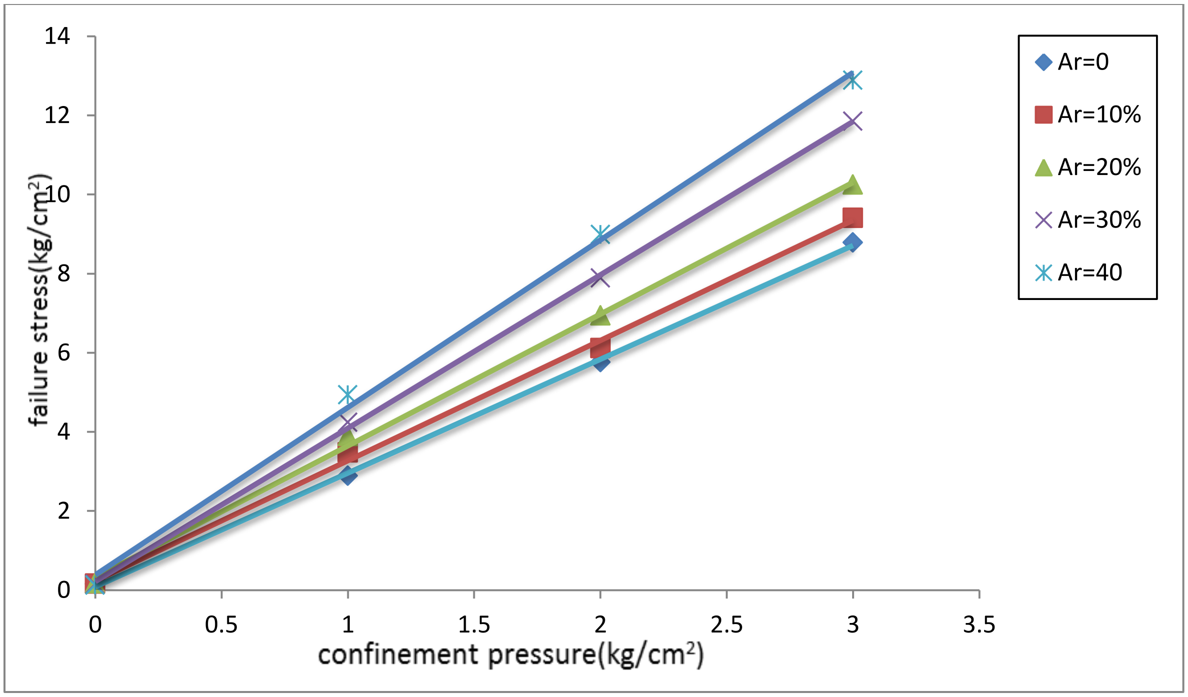


Fig 4.30: variation of failure stress in 0.25 length reinforced pond ash in different confinement pressure

4.3.3 BEARING CAPACITY OF STONE COLUMNS

4.3.3.1 Load settlement behavior

Footing load tests were carried out on untreated pond ash specimens compacted to their corresponding MDD and OMC .This test was carried out to study the load settlement behavior of pond ash reinforced with stone column in different length ratio of their respected area ratio and the test result and behavior has plot in Fig-4.31, 4.32, 4.33, 4.34.From these graph it is observing that the by increase of length ratio from 0.25 to 1the failure stress of varying area ratio 10,20,30,40% is 2.844 to 4.124 kg/cm^2,3.26 to 4.868 kg/cm^2,4.133 to 6.234 kg/cm^2 and 4.767 to 7.841 kg/cm^2 respectively. From the graph it can be concluded that for each length ratio the failure stress increases linearly with the area ratio. With the decrease in the length ratio, the failure strain is observed to be increasing. This is due to the fact that, for the case of higher length ratio the stone column- having a higher angle of friction and higher density- leads to a lower strain. For the case of low length ratio, the particles of the stone column and the pond ash settle on application of the load. However, since pond ash forms a major portion of the specimen, the strain caused is higher than for the larger length ratios. It shows higher stress for higher area ratios. Similarly higher stresses for a particular area ratio were observed for higher length ratios. Because of the higher angle of internal friction it has, stone column plays a major part in increasing the strength of pond ash. From the Fig it is visible that the initial stress is maximum at higher area ratio and for a particular area ratio, the initial stress increases linearly with the increase of length ratio. Also, the maximum failure stress depends on the maximum area ratio and length ratio. After reaching the maximum failure stress, the failure zone rises to the upper surface of pond ash bed as shown in Fig-4.35.

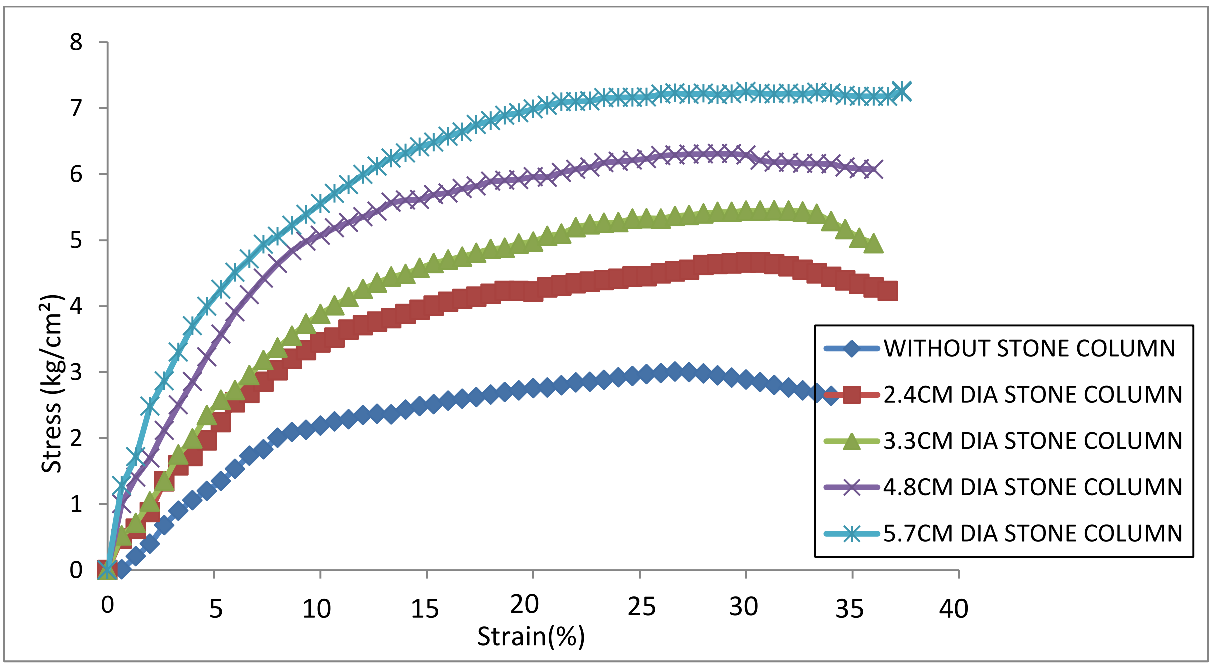


Fig 4.31: variation of failure stress and settlement in full length reinforced pond ash

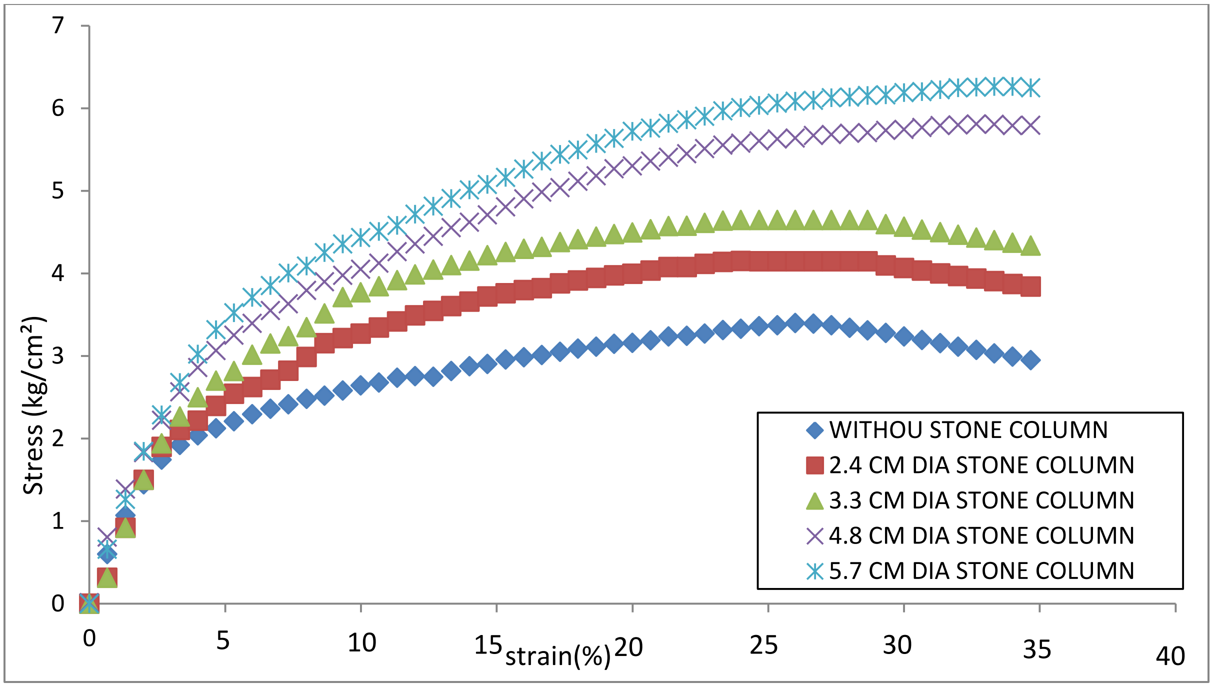


Fig 4.32: variation of failure stress and settlement in 0.75 length reinforced pond ash

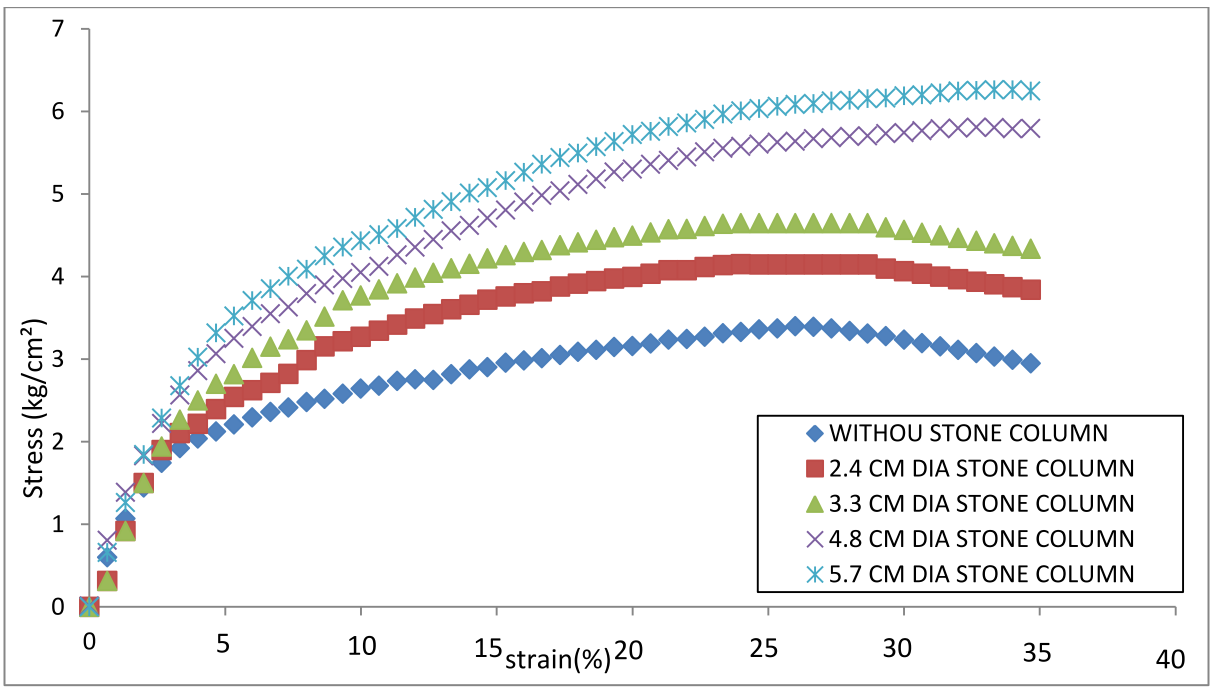


Fig 4.33: variation of failure stress and settlement in 0.5 length reinforced pond ash

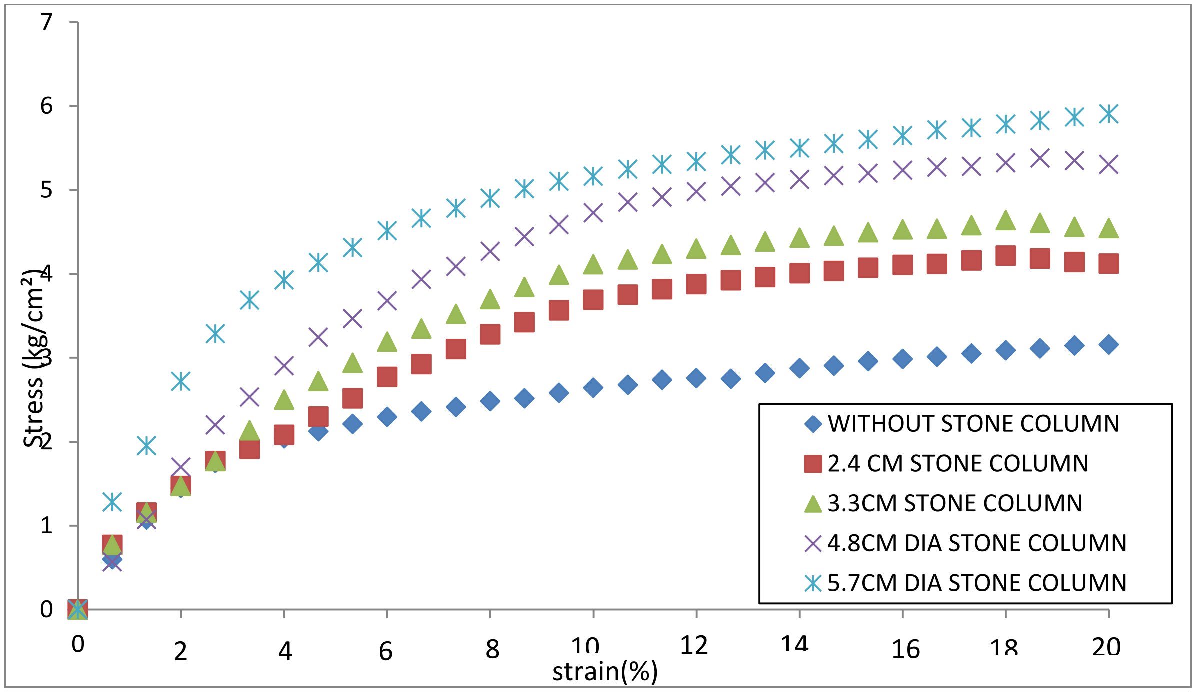


Fig 4.34: variation of failure stress and settlement in 0.25 length reinforced pond ash

Fig 4.35 Failure pattern at compacted pond ash bed

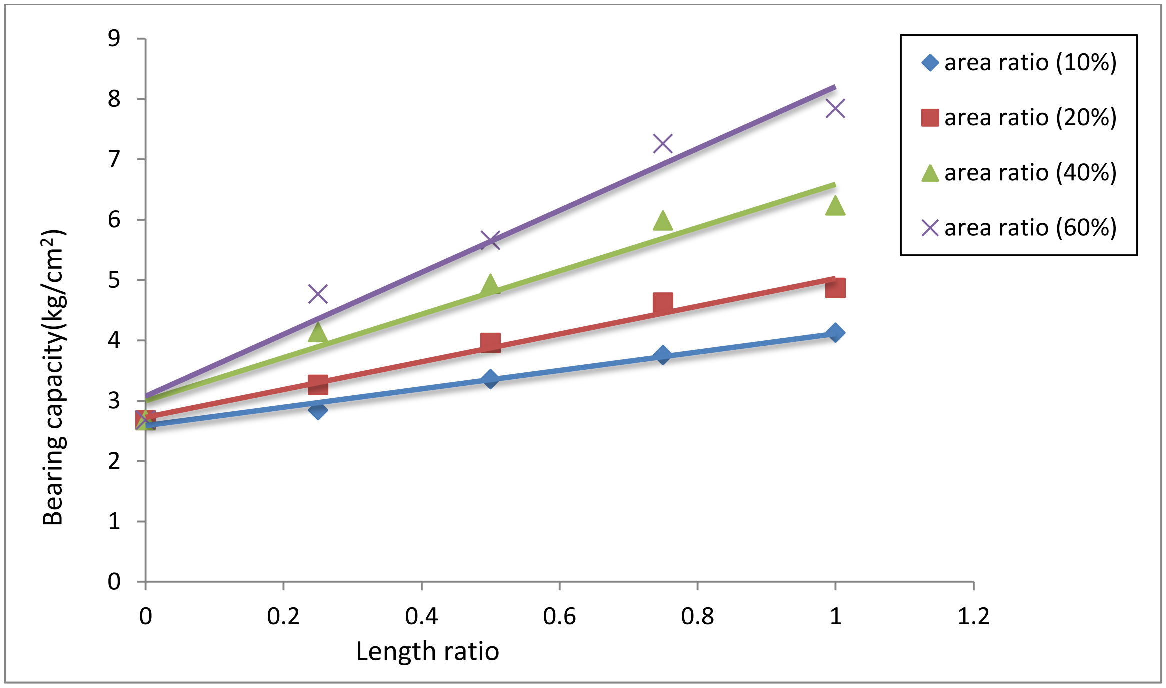


Fig 4.36 Variation of bearing capacity with length ratio

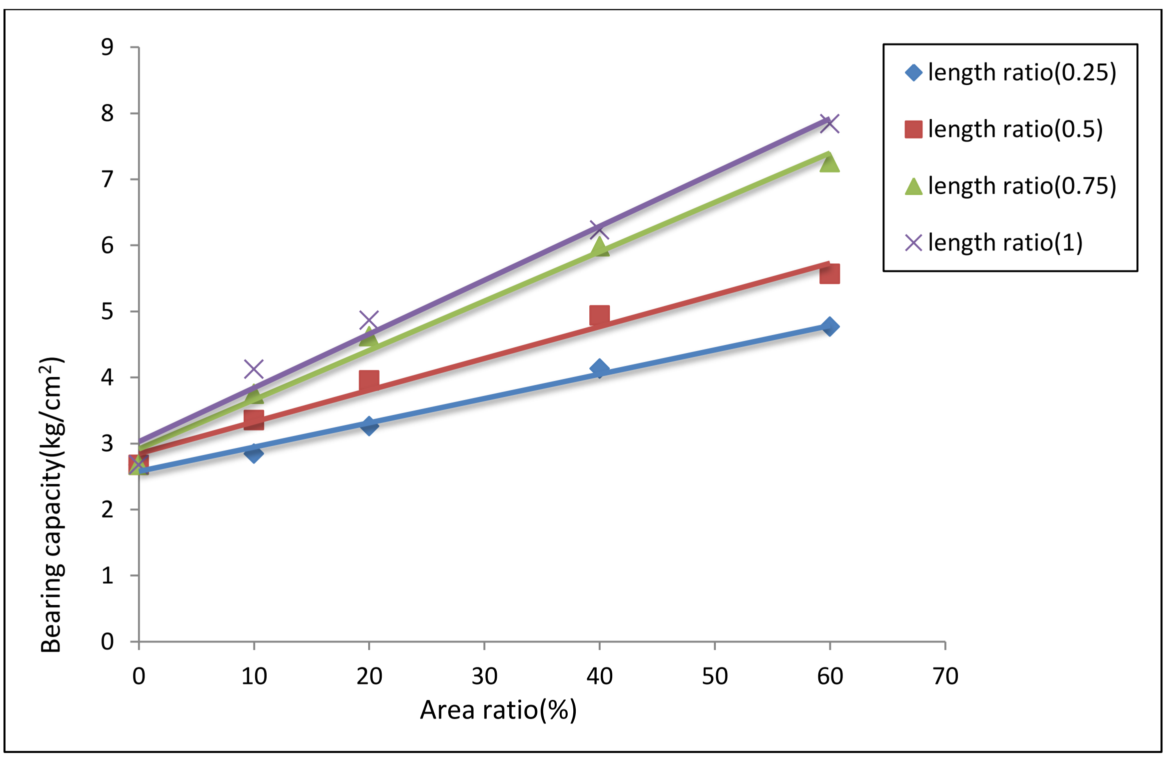


Fig 4.37 Variation of bearing capacity with area ratio

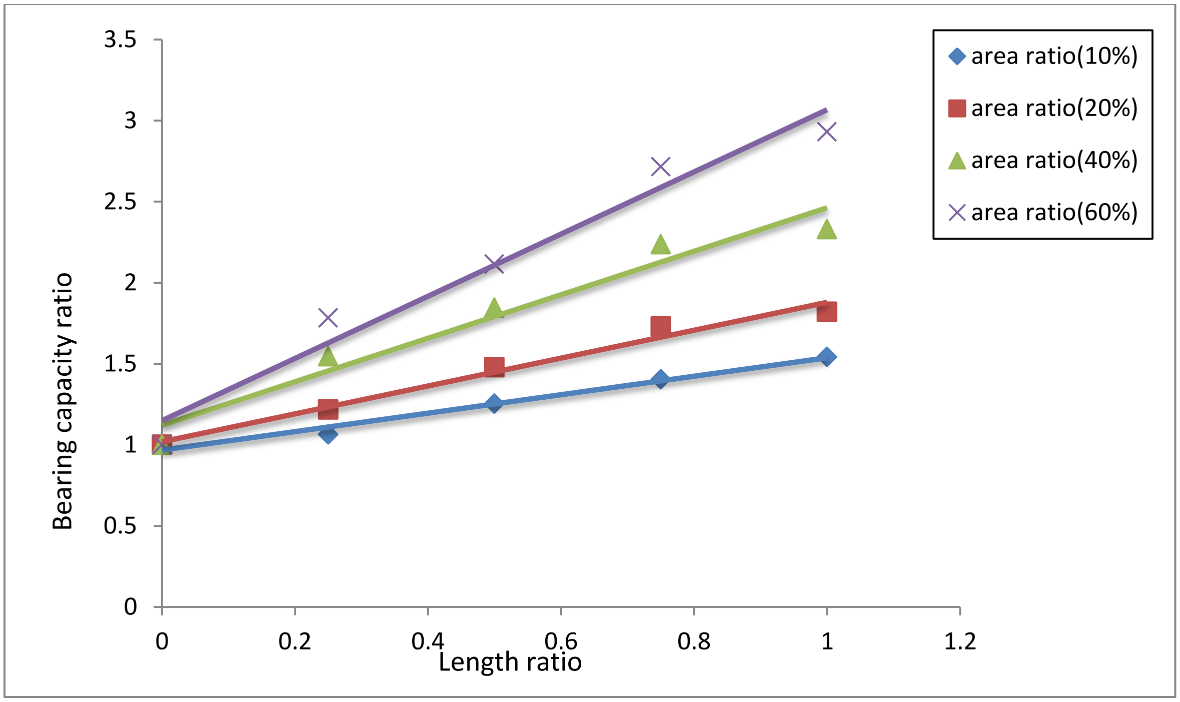


Fig 4.38 Variation of bearing capacity ratio with length ratio

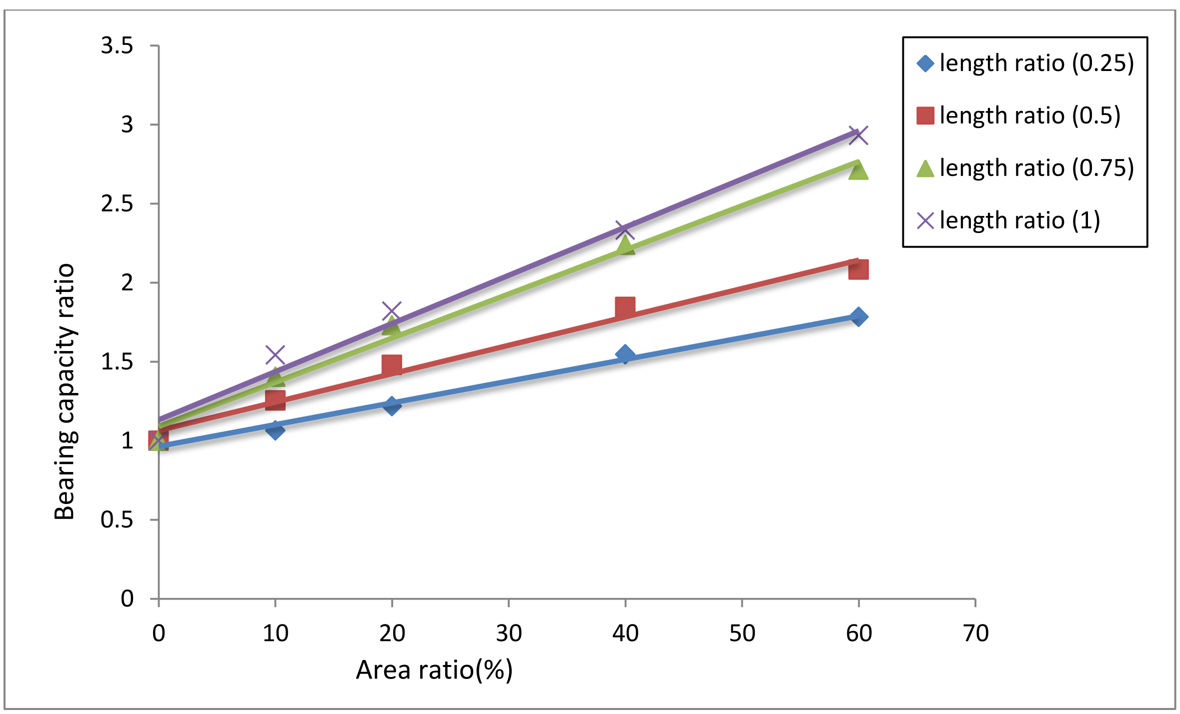


Fig 4.39 Variation of bearing capacity ratio with area ratio

From the Fig-4.36 and 4.37 it is observe that with the increase of length ratio to their respected area ratio bearing capacity increases linearly. It shows that with the increase of stone column diameter and reinforcing length due to high compacted density the frictional angle increases linearly in result bearing capacity also increases linearly. The stone column diameter and length plays a major part in increasing the bearing capacity of stone column.

In the Fig-4.38 and 4.39 there has mention bearing capacity ratio which is the ratio of bearing capacity of reinforced pond ash to bearing capacity of without reinforced stone column. From that fig it is observe that as the increase of length ratio of the respected area ratio bearing capacity also increases but at 0.75 length ratio reinforced pond ash has more effective as compare to other if considering the use of materials. At full length reinforced pond ash shows bearing capacity is closer to 0.75 length ratio reinforced pond ash but requirement of material is more.

CHAPTER 5

CONCLUSIONS

CONCLUSION

Based on the tests conducted on the pond ash collected from RSP Rourkela and model footing loading tests conducted on compacted pond ash beds reinforced with stone columns of different area ratios and length ratios the following main conclusions are drawn:

- The pond ash consists of grains mostly of fine sand to silt size with uniform gradation of particles. The percentage of pond ash passing through 75µ sieve was found to be 18.84%. Coefficient of uniformity (Cu) and coefficient of curvature (Cc) for Fly ash was found to be 6.13 & 2.61 respectively, indicating uniform gradation of samples. The specific gravity of particles is lower than that of the conventional earth materials.
- An increase in compaction energy results in closer packing of particles thus increase in dry density whereas the optimum moisture content decreases.
- Dry unit weight of compacted specimens is found to change from 0.984 to 1.23 gm/cm^3 with change in compaction energy from 119kJ/m^3 to 2674 kJ/m^3, whereas the OMC is found to decrease from 43.23 to 31.7 %. The low compacted density may be attributed to the rounded shape of particles, uniform gradation of the sample and the low specific gravity of the constituent particles.
- Pond ash possesses low unit cohesion. But both the unit cohesion and frictional angle is found to increase with increase in compaction energy. The increase in frictional angle is attributed to closer packing and interlocking of particles.
- A linear relationship is found to exist between the compaction energy and unconfined compressive strength. The UCS value is found to change from 19.587 to 66.758 kPa with change in compaction energy from 119kJ/m^3 to 2674kJ/m^3 indicating that the gain in

strength is not so remarkable. It revealed from the test results that a linear relationship exists between the initial tangent modulus with unconfined compressive strength and deformation modulus.

- The UCS value at saturation condition is found to change from 8.142 to 38.45 kPa with change in compaction energy from 119kJ/m^3 to 2674kJ/m^3.These values are much lower compared to the values obtained at OMC. This indicates that saturation of pond ash specimens results in drastic reduction of strength.
- The triaxial test results shows that as the increase of confining pressure the stress value is also increasing linearly. The test which was conducted between sample prepare at different compactive energy, maximum stress show the sample prepare at higher compactive effort. Due to the confinement and sample prepared at higher compactive effort attributed to the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component and angle of internal friction in the sample.so that the unit cohesion was increased from 0.106 kg/cm^2 to 0.239 kg/cm^2 and angle of internal friction was increased from 19.87^0 to 37.4^0.
- The UCS tests among all area ratio and their respected length ratio of reinforced stone columns as increasing of area ratio of reinforced pond ash the stress value has decreased with the decreased of strain. At area ratio 10% it has observed that stress value was increasing by increase order of length ratio of stone column whereas at 40% area ratio it was showing reverse. It was due to adequate amount of confining pressure was not sufficient to keep stable sample prepare at 40% area ratio.

- The triaxial tests in different area ratio of their respected length ratio with different confining pressure, the higher area ratio of their respected length ratio shows maximum stress due to confinement. The stress value was increased by increase of confining pressure. So here due to full length of stone column and confining pressure the stone column show more effective as compare to other because of the closer packing of particles, resulting in the increased interlocking among particles. A closer packing is also responsible in increasing the cohesion component and angle of internal friction in the sample.
- In the footing load test the failure stress increases linearly with the area ratio. With the decrease in the length ratio, the failure strain is observed to be increasing. This is due to the fact that, for the case of higher length ratio the stone column- having a higher angle of friction and higher density- leads to a lower strain.
- For the case of low length ratio, the particles of the stone column and the pond ash settle on application of the load. However, since pond ash forms a major portion of the specimen, the strain caused is higher than for the larger length ratios.
- It shows higher stress for higher area ratios. Similarly higher stresses for a particular area ratio were observed for higher length ratios. Because of the higher angle of internal friction it has, stone column plays a major part in increasing the strength of pond ash.
- The initial stress is maximum at higher area ratio and for a particular area ratio, the initial stress increases linearly with the increase of length ratio. Also, the maximum failure stress depends on the maximum area ratio and length ratio. After reaching the maximum failure stress, the failure zone rises to the upper surface of pond ash bed.

- It is observe that with the increase of length ratio to their respected area ratio bearing capacity increases linearly. It shows that with the increase of stone column diameter and reinforcing length due to high compacted density the frictional angle increases linearly in result bearing capacity also increases linearly. The stone column diameter and length plays a major part in increasing the bearing capacity of stone column.
- For the effective and economic purpose it is observing that the increase of length ratio of the respected area ratio bearing capacity also increases but at 0.75 length ratio reinforced pond ash has more effective as compare to other if considering the use of materials. At full length reinforced pond ash shows bearing capacity is closer to 0.75 length ratio reinforced pond ash but requirement of material is more.

CHAPTER 6

SCOPE FOR FUTURE WORK

SCOPE FOR FUTURE RESEARCH

In the present work model tests were carried out on compacted pond ash beds reinforced with stone columns of different length ratio and area ratios. The test results are very encouraging. However, the following few aspects are to be studied before this technique is actually applied in the field.

SCOPE OF FUTURE RESEARCH

- Field test on large size footings / prototype test be carried out to validate the findings of model test results
- Test should be carried out on group of stone columns loaded simultaneously
- Behaviour of jacketed and anchored stone columns be studied
- Liquefaction susceptibility of the system to be studied
- Studies on stone columns with horizontal reinforcement

CHAPTER 7

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